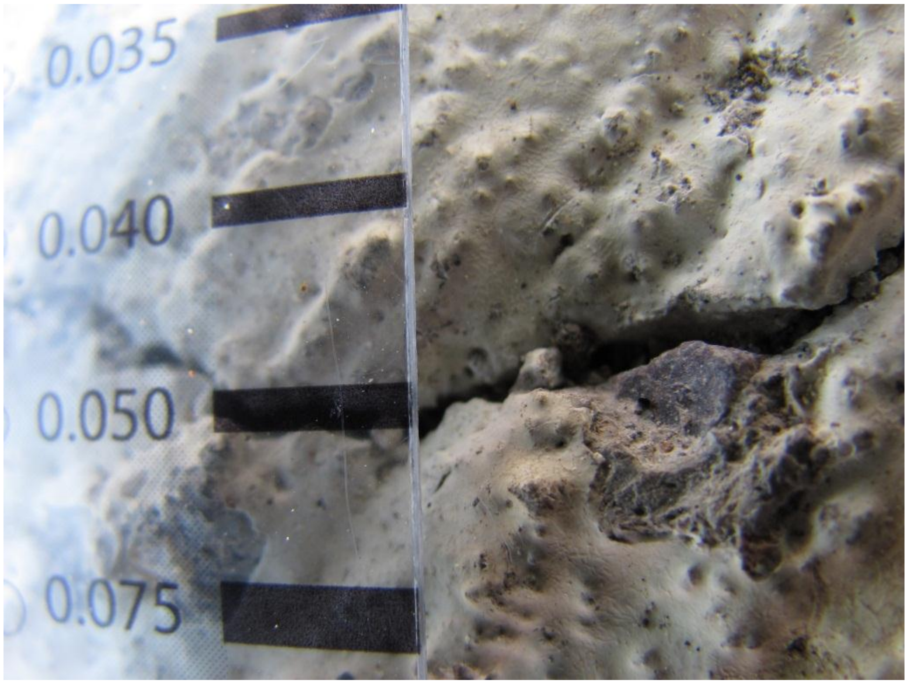
TASK ORDER NO. 1
ON-CALL STRUCTURAL CONCRETE BRIDGE DECK CRACKING INVESTIGATION SERVICES
WJE No. 2009.2643

Final Report



Contract No.: 59A0713
Task Order No.: 1
Consultant Firm: Wiss, Janney, Elstner Associates, Inc.
Report Date: October 26, 2011

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Caltrans

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EXECUTIVE SUMMARY

Wiss, Janney, Elstner Associates, Inc. (WJE) performed an investigation of early-age bridge deck cracking for the California Department of Transportation (Caltrans) under Task Order No. 71301, Contract No. 59A0713. The objectives of this investigation are to recommend changes in current Caltrans design and construction specifications, and construction procedures to mitigate early-age cracking in bridge decks in California.

Early-age cracking in concrete bridge decks is a serious and pervasive problem in California and throughout the U.S. It is the single most prevalent deck distress reported by state departments of transportation (DOTs). Although numerous studies and significant research has been performed to identify the causes and recommend solutions to early-age deck cracking, the problem still persists.

The research provided in this report relates specifically to providing practical and realistic recommendations to reduce bridge deck cracking in California with concretes using local materials. This report summarizes the findings of the investigation. The scope of the investigation included the following:

1. Literature review
2. Review of measures taken by other DOTs to reduce early-age cracking
3. Review of Caltrans Specifications/Design Practice/Design Policies
4. Field and laboratory testing of two newly constructed bridges
5. Analytical studies using finite element (FE) models and lattice modeling
6. Validation of potential solutions
7. Recommendations to Caltrans

Our research identified a number of changes to current Caltrans design and construction specifications, and construction procedures that should help mitigate early-age cracking in bridge decks. These changes have been validated by previous studies and research, by field and laboratory testing and analytical studies that were conducted as part of this investigation, and by experts in research, construction, and design that were surveyed as part of this investigation.

BACKGROUND OF EARLY-AGE CONCRETE CRACKING

Early-age cracking of bridge deck concrete is a nationwide problem that was researched by WJE in NCHRP Report 380 "Transverse Cracking in Newly Constructed Bridge Decks" published in 1996 [Krauss and Rogalla, 1996]. This cracking usually occurs within the first six to twelve months after construction but can continue for several years. Typical 0.010 to 0.015 in. (0.25 to 0.38 mm) wide cracks, extending through the thickness of the deck, transversely spaced 3 to 10 feet (0.9 to 3.0 m) apart are common characteristics of these cracks. Leakage of deicer solutions through these cracks are a primary cause of premature corrosion of deck reinforcement and supporting beams. Figure 1 shows typical deck cracks that are nationally problematic and characteristic of cracking that is the focus of this research. Identifying the causes and preventing transverse cracking in bridge decks is difficult and complex. There are also other types of cracking that can occur on bridge decks that can affect appearance and performance.

Figure 1. Underside of a deck showing leakage through full depth transverse deck cracks

General Introduction to Cracking in Bridge Decks

Several effects can cause cracking in concrete decks, including plastic shrinkage, crazing, settlement, autogenous shrinkage, drying shrinkage, and thermal shrinkage. A general introduction of these crack types follows.

Plastic Shrinkage Cracking

Plastic shrinkage cracks, as shown in Figure 2, are those that occur while the concrete is still fresh and has not fully hardened. They appear on the exposed horizontal surfaces and can occur anytime ambient conditions (air temperature, concrete temperature, humidity, and wind velocity) are conducive to rapid evaporation. Plastic shrinkage cracking occurs when the rate of surface evaporation exceeds the rate of bleeding of the concrete. Plastic shrinkage cracks are roughly parallel to each other, randomly spaced, and not directly in-line with reinforcement. The width of the crack at the surface may be large (typically 0.02 in. to 0.03 in. (0.51 to 0.76 mm) but the cracks are usually no more than 2 or 3 ft. (0.6 or 0.9 m) long and are rarely more than 2 to 3 in. (50 to 75 mm) deep. Such cracks are seldom significant structurally, and once the crack starts, the stresses rapidly dissipate. Plastic shrinkage cracks can be severe and aesthetically unacceptable. Plastic shrinkage cracks can also allow the penetration of water, deicers, and atmospheric gases into the concrete promoting local areas of reinforcing corrosion and can worsen cracking from thermal, drying, or load-induced stresses.

Concretes most susceptible to plastic shrinkage cracking have little bleed water, usually due to low water contents, high paste, low water/cement ratios, or the use of latex or polymer modifiers, superplasticizers high range water reducers, (HRWRs), air entrainment, or silica fume. HRWRs reduce the water content and therefore the bleeding capacity of the concretes. Silica fume intensifies the problem because of the extreme fineness of the fume material. The HRWR reduces the amount of bleed water available while the high fineness the silica fume reduces the rate at which the water can move through the concrete.

If the rate of evaporation from the surface exceeds the rate of bleeding and additional water is not added through moist curing, surface tension and tensile stresses form on the concrete surface and cracking can occur. Reducing the evaporation rate or increasing the bleeding capacity of the concrete will prevent plastic cracking. The former can be accomplished to various degrees by sunscreens, windbreaks, fog mist, monomolecular curing films and immediate wet curing. The most effective means of avoiding the loss of bleed water (reducing evaporation) is with fogging during construction, followed by rapid placement of wet curing and impermeable curing covers such as polyethylene sheeting. Increasing the bleeding capacity of the concrete is usually not practical or desirable.

Figure 2. Cracks characteristic of plastic shrinkage cracking that occurred on the Markham Ravine Bridge.

Craze Cracking
Craze cracking is a network of fine surface cracks caused by shrinkage of the mortar on the very top surface of the deck. The cracks rarely penetrate more than about 0.3 in. (7.6 mm) into the concrete and are most obvious on steel troweled surfaces. While they occur at a very early age, they are usually not readily visible until after the curing period when the surface of the concrete is allowed to dry. Craze cracking does not affect the structural integrity of the concrete and rarely affects the durability or wear. Crazing occurs due to poor construction practices such as inadequate curing, an excessively high water content or bleeding, adding water to the surface to improve finishing, excessive floating, or finishing the concrete while bleed water is present on the surface. Crazing can be prevented by proper construction practices and by starting proper wet curing as soon as possible. Being superficial, crazing cracking will not be discussed in any depth in this report.

Settlement Cracking
Chairs rigidly support deck reinforcement. Excess water in plastic concrete allows aggregate and cement particles to be maintained in suspension while concrete is transported and consolidated. After placement, these solids settle and water bleeds. Supported reinforcement stops the solids from settling and can cause tensile stresses and cracking directly over and in-line with the reinforcement leaving small voids under the bar, as illustrated in Figure 3. Settlement cracks are particular detrimental since they expose lengths of bars to atmospheric and deicer induced corrosion and can promote full depth cracking when combined with later-age drying and thermal contraction. The risk of settlement cracking over bars increases with lower bar cover, higher concrete slump, and larger bar diameter. Settlement cracking can be avoided by placing the concrete at a low slump and using effective consolidation procedures. Increasing concrete cover and reducing bar size also reduce the risk of settlement cracking.

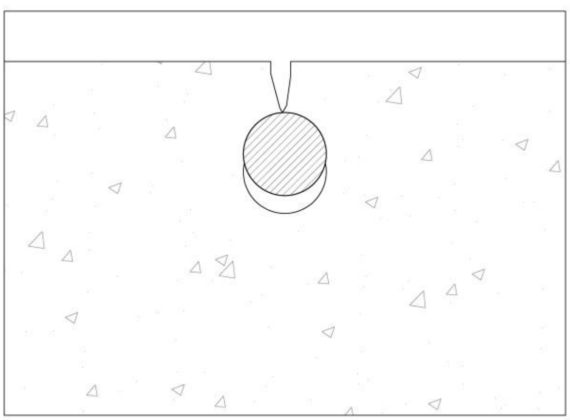


Figure 3. Schematic showing effect of supported bar in causing cracking in the concrete surface over the bar due to concrete settlement. Note potential notch effect and void under bar that may promote further cracking due to drying or thermal contraction.

Autogenous Shrinkage
Volume change occurs during cement hydration (the chemical reaction of the cement). Uptake of water during hydration can lead to expansion; however, if excess water is not available shrinkage occurs. The first large stresses in a new concrete deck usually develop during the first 12 to 24 hours, when autogenous shrinkage occurs and concrete temperatures change rapidly during early hydration. Altoubat

and Lange [2001] found the rate of shrinkage during the first 50 hours to be highest due to contributions of autogenous shrinkage. Concrete mixes having very low water/cement ratio or fine mineral additives, such as silica fume, can have high autogenous shrinkage. Autogenous shrinkage is often considered part of drying shrinkage and might be only about 40 microstrain shrinkage at 28 days (about 10 percent of the total drying shrinkage). However, it occurs at a time when concrete is most susceptible to cracking due to the loss of hydration heat and the end of moist curing periods. High temperatures, high cement content, cements with high C3A and C4AF content, and possibly very fine cements tend to increase autogenous shrinkage (Neville, 1981).

Concrete Drying Shrinkage

As concrete bridge decks dry, they shrink. The drying occurs differentially throughout the deck so the maximum drying and tensile stresses occur at the drying surfaces. The loss of mix water from newly cast concrete during exposure to air at less than 100 percent relative humidity (RH) causes drying shrinkage. Starting from the saturated condition, the relation between the amount of water lost and unrestrained drying shrinkage of new concrete is roughly linear through two distinct phases. In the first phase, the water that is lost consists primarily of free water. It is accompanied by a relatively small amount of shrinkage. During the second phase, the water loss consists primarily of adsorbed water that is accompanied by a large amount of shrinkage. Adsorbed water is contained in capillary and gel pores. This part of the drying shrinkage is irreversible so the shrinkage strain of a concrete dried for the first time will be the highest. (Neville, 1981). Drying shrinkage of unreinforced, unrestrained bridge deck concrete in a 50 percent RH environment might range from about 500 to 1000 microstrain (500 to 1000 x 10^{-6} in./in.). Higher or lower RH, periodic wetting, and the concrete mix can significantly lower or increase the shrinkage.

Drying and shrinkage continues until the concrete moisture is in equilibrium with its environment. Therefore, bridge decks cast in areas of California that are drier will have more total drying shrinkage. These areas also often have larger temperature fluctuations further aggravating shrinkage stresses. The drier the environment, the more water that is lost from the concrete and the greater range of pore sizes that are being emptied. The larger pores empty first followed by consecutively smaller pores until the internal menisci are in equilibrium with the surrounding relative humidity. (ACI 231R-10, 2010). The relationship between free, unrestained shrinkage and the probability of cracking is not linear and a small reduction in shrinkage may have a substantial benefit in reducing cracking. (Radlinska and Weiss, 2006).

The rate of water loss from the concrete depends on the evaporation rate and the surface-to-volume ratio of the element. Within the concrete, the rate of moisture loss varies inversely as the square of the distance from the nearest drying surface. High surface-to-volume ratio elements, such as bridge decks, result in faster drying and shrinkage. With rapid drying rates combined with very slow diffusion of mix water toward surface, due to low concrete permeability, a large degree of strain differential can develop. This is a result of the surface drying and shrinking, while the interior portions of the deck remain at a high moisture content and therefore shrink much less. This can produce tensile stress near the surface and can be additive to the overall drying shrinkage stresses caused by reinforcing and girder restraint. Once concrete has reached equilibrium with a given temperature and RH, its volume will remain stable until the humidity or the temperature changes. Actual bridge decks never reach a uniform equilibrium due to cyclic rain and drying and diurnal temperature fluctuations.

In 1942, Carlson [1942] stated that there are seldom any conditions to be fulfilled in designing concrete for low shrinkage that are not already fulfilled for other reasons since concrete which gives the best quality and economy also gives the lowest shrinkage. However, with the development of chemical admixtures, modern concretes can achieve adequate slump and workability to be pumped and consolidated, all with little consideration of basic concrete design principles such as aggregate quality, aggregate gradation, paste content, or paste quality; many of the concrete characteristics Carlson recognized had an effect on cracking. Reducing cracking in bridges decks may be well served by recognizing and using fundamental concrete design to reduce concrete shrinkage with less reliance on modern chemical admixtures. Concrete drying shrinkage is affected primarily by the concrete paste content, cement and supplementary cementitious materials, aggregate gradation, aggregate type, curing methods, and use of shrinkage-reducing admixtures.

Thermal-Induced Cracks

The first large stresses in a new concrete slab usually develop during the first 12 to 24 hours, when the autogenous shrinkage occurs and concrete temperatures change rapidly during early hydration. Cement hydration is exothermic so the temperature of the concrete increases as it is setting and gaining strength. Stresses develop as the concrete cools. Reducing the peak concrete temperatures during this cycle will reduce early stresses. This can be done by selecting cement with low heat, placing concrete during cooler weather (such as during the evening or at night), placing cool concrete, and misting the concrete during placement and wet curing.

The coefficient of thermal expansion (CTE) of concrete varies based on aggregate type, concrete age, and moisture content. The thermal coefficient for concrete is very high for the first 6 hours but decreases rapidly over the first 24 hours as the concrete cures. (Hedlund, 1996; Byfors, 1980). The CTE of concrete was found to be near the same when concrete is dry or when saturated but there is a dramatic increase when the concrete is at intermediate relative humidity (RH). (Meyers, 1950; Zoldners, 1971). A fully saturated pore system and a fully dried pore system may give lower values of CTE of around 5.6 to 6.7 x 10^{-6} /°F (10 to 12 x 10^{-6} /°C) while a partially saturated pore solution may show maximum values near 13.9 x 10^{-6} /°F (25 x 10^{-6} /°C). (ACI Committee 231, 2010). This is important since bridge deck concrete is typically at intermediate RH and in the drying state when cracking occurs.

Accumulation of shrinkage and thermal stresses causes most early-age cracks in bridge decks. The thermal expansion and contraction of the deck concrete depends significantly on the concrete aggregates and curing. Aggregate composition can change the thermal coefficient of expansion of concrete from about 4.0 to 7.5 x 10^{-6} /°F (7.0 to 13.5 x 10^{-6} /°C). Caltrans aggregates tend to have thermal coefficients on the higher side of this typical range so concretes will expand and contract more when subjected to a certain temperature change, increasing the risk of cracking. Thermal shock can also aggravate deck cracking if say a cold rain contacts a warm deck surface. Thermal shock resulting in temperature differences between the concrete surface and core of 90°F (50°C) has been reported to cause cracking. (Neville, 1981), p.378.

Restraint

Unrestrained concrete expands when heated, contracts when cooled, and shrinks as it dries. These thermal and shrinkage movements are expressed in terms of strain. Strain by itself does not necessarily cause stress that is necessary for cracking. When concrete undergoes a uniform or linearly distributed shrinkage or temperature change, it will not develop stresses if it is not physically restrained against movement. However, if restrained, the force or pressure restraining the concrete causes stress.

Shrinkage and temperature stresses develop in all bridge decks, because the supporting girders or underlying box girder sections restrain the natural thermal and shrinkage movement of the deck. When the deck and girders consist of different materials (steel and concrete, or different concretes) with different thermal expansion rates, even a constant temperature change will cause stresses because the different materials expand differently and cannot expand freely where they are attached. Further, temperatures are rarely uniform or linearly distributed, and shrinkage is also nonlinearly distributed. Nonlinear shrinkage and temperature changes cause stress, even without an external source of restraint.
The structural elements below the deck restrain the thermal and shrinkage strains in the deck, causing stresses in all elements. This restraint develops from friction between the concrete and its supporting girders and from mechanical shear transfer through any studs or reinforcement across the interface of the deck and the underlying structure. To a lesser extent, steel reinforcement embedded in the deck also restrains the deck against shrinkage and against thermal movements when the reinforcement has a different coefficient of thermal expansion than the concrete.

The amount of shrinkage and thermal restraint against a deck depends largely upon the stiffness characteristics of the underlying structure. When the underlying structure is very stiff relative to the deck, the underlying structure can restrain much of the thermal and shrinkage movement that would develop in the deck if it was unrestrained. On the other hand, flexible underlying structural elements will restrain less deck movements. If the concrete deck would have a free shrinkage of 500 microstrain if unrestrained, but restraint allows it to shorten only 200 microstrain, the restraint would be 60 percent (60 percent of the 500 free microstrain, or 300 microstrain, was restrained).

Why Do We Care?

Deck cracking can affect corrosion of embedded reinforcement in a very adverse way if the deck is subjected to deicer chemicals or seawater spray. Kansas (Lindquist, Darwin and Browning, 2005) found that chloride concentrations taken at crack locations often exceeded the corrosion threshold of black (uncoated) steel after only one winter season. Conversely, chloride concentrations taken from uncracked locations rarely exceeded the corrosion threshold at the bar level even after ten to fourteen years. Even if the bridge is not subject to deicers, water and carbon dioxide penetration through cracks will aggravate corrosion of the embedded reinforcing steel and supporting girders. Further, water leakage through cracks causes leaching and is unsightly. It is clear that attention should be focused at preventing early-age deck cracks and repairing cracks in a timely manner should they occur.

Summary

A combination of autogenous/drying shrinkage and thermal stresses cause most early-age cracking in bridge decks, but other conditions can also cause or contribute to deck cracking. Examples of other causes include early plastic shrinkage in the fresh concrete due to excessive surface evaporation. Additionally, settlement cracks expose top mat reinforcing and the crack and associated void directly below the bar can create notch effects that allow full depth cracks to initiate and more easily propagate. Falsework settlement or deck deflections can cause cracking over the piers or supporting girders; however, these cracks are localized and not typical of the widespread cracking seen across most decks.

Careful attention to the concrete mix design, and to placing, finishing, and curing practices, can reduce the risk and severity of deck cracking. However, some cracking may be unavoidable in reinforced decks having high restraint conditions, including many bridge types commonly built by Caltrans.

Introduction to Research

A bibliography of the research into early-age bridge deck cracking provided in this report is not exhaustive but relates specifically to this project for providing practical and realistic recommendations to reduce bridge deck cracking on California bridge types with concretes using local materials. Practical application of the current knowledge was the focus of the literature review. A few of the primary publications that summarize the various factors related to early-age deck cracking are discussed below as further introduction to the problem. DOT specific research is discussed in the report section for the review of measures taken by other DOTs to reduce early-age cracking.

Carlson

Cracking of concrete has been a concern for a long time. Roy W. Carlson [1942] presented a paper to the Boston Society of Civil Engineers on the "Cracking of Concrete" due to shrinkage from drying in 1942. He stated that drying shrinkage contributes to the cracking of nearly all concrete, especially thin sections in dry climates, and that it is less of a factor in massive sections or on structures in moist climates. He noted that drying shrinkage alone or temperature change alone would not cause cracking, but that in thin sections drying shrinkage needs the help of a quick temperature drop while in thick sections the temperature change often needs the help of surface drying to start cracks.

Carlson described the difference between the tendency of concrete to shrink and its tendency to crack, with the tensile stress being the product of three terms: free shrinkage, effective modulus of elasticity, and degree of restraint. Effective modulus is also called sustained modulus in that it includes the plastic flow or creep, and is softer than the modulus of elasticity under short-term loading. He defined the degree of restraint as the percentage of the free shrinkage that the restraint prevented, and noted that this restraint can be due to the external conditions (beams, girders or piers) or internal conditions (uniform concrete compositions with quicker surface drying, restrained by the slower drying underlying concrete).

Carlson recognized reducing concrete shrinkage was most desirable. This is done by limiting paste content and total water contents, using the largest size of well-graded aggregates, and using sound aggregate that has low shrinkage characteristics. Paste content is the fraction of concrete that does not contain fine or coarse aggregate. Carlson reports that roughly speaking, cement paste will shrink about half of one percent in length and concrete will shrink about one-tenth this amount, or 0.05 percent in length, even though about one-third of the concrete is cement paste. Examination of a cross section of concrete helps explain this discrepancy as paths can be drawn almost straight from end to end along which less than one-twentieth is cement paste. Carlson believed that internal restraint from aggregate explains the reduced shrinkage, highlighting the importance of a proper aggregate gradation and aggregate quality in reducing shrinkage while obtaining proper concrete workability.

Portland Cement Association (PCA)

In 1970, the Portland Cement Association (PCA, 1970) performed an extensive study of bridge deck cracking and distress. The study included detailed field evaluations of seventy bridges in four states, and random surveys of over one-thousand bridges built between 1940 and 1962 in eight states. The randomly surveyed bridges had scaling due to non-air entrained concrete and various types of cracking. About two-thirds of the bridges had deck cracking, with most deck cracks being transverse to the span. These transverse cracks appeared to increase with age and span length, and had a higher incidence for continuous-span bridges and decks supported on steel girders. The close-interval cracks occurred above the transverse reinforcing steel and were reported to be mainly caused by subsiding plastic concrete. Recommendations from this study included using the largest maximum size aggregate to minimize paste content, and using a maximum slump of 3 in. (75 mm) to reduce the effects of bleeding, shrinkage, and risk of settlement cracking.

Krauss and Rogalla

Since we published NCHRP Report 380 "Transverse Cracking in Newly Constructed Bridge Decks" in 1996, large amounts of research and literature have been published regarding concrete cracking in bridge decks. This literature and experience point to key factors that influence deck cracking. Many of the findings and recommendations provided in the NCHRP Report 380 have been tested by others both in the laboratory and by many field studies, and in whole have been found to be properly focused.

NCHRP Report 380 investigated the causes of deck cracking by literature review, DOT staff surveys, instrumentation of a bridge deck replacement project, analytical studies of the stresses resulting from different combinations of variables thought to influence cracking, and laboratory testing to evaluate the cracking tendency of various mix designs. Stresses due to thermal and shrinkage were calculated for more than 18,000 combinations of bridge geometries and material properties. The study found that girder size and spacing, as well as deck thickness, influenced deck cracking. It also found that material properties, including shrinkage and thermal properties, influenced deck stresses more than geometry or other design parameters.

The study found that the cracking tendency increased with increasing cement content and decreasing water/cement ratio, and that silica fume increased the cracking tendency. Aggregate type was found to influence cracking tendency, with concrete made with lightweight and crushed aggregates being more crack-resistant than concretes containing rounded river gravels, such as often found in California.

To reduce deck cracking, NCHRP Report 380 recommended minimizing the cement contents (not exceeding 517 lb. /cu yd.), using an optimized aggregate gradation with a large-aggregate size of 1.5 in. (38 mm), using concrete mixes with low shrinkage characteristics and moderate rates of strength gain, and casting decks in the late afternoon or evening to reduce the high concrete temperatures during hydration and the rate of thermal contraction as hydration heat dissipates.

In a survey of DOTs, DOT engineers expressed concern that the deck cracking increased in severity in about the mid-1970's. This coincided with several significant changes in the AASHTO specifications for bridge deck concrete, as shown in Table 1. The changes included an increase in the minimum cement content from 6.0 to 6.5 sacks per cu. yd., an increase in the minimum compressive strength at 28 days from 3,000 psi to 4,500 psi (20.6 MPa to 31 MPa) for air entrained (AE) concrete or to 4,000 psi (27.5 MPa) for non-air entrained (A) concrete, and a maximum water/cement ratio requirement of 0.455(AE) or 0.490(A) was specified. Also, the slump requirements for the deck concrete were dropped. The increase in cement content, lower w/c, and increased strength, as well as an allowed increase in slump could all contribute to the increased risk of early-age cracking in bridge decks.

Table 1. Summary of AASHTO Specifications for Bridge Deck Concrete

AASHTO Concrete Specifications						
Year	**Class**	**Strength (psi)**	**W/C**	**Bags/Yd^3**	**Air**	**Slump**
1931	A	3000	--	6.0	--	2-3
1941	A	3000	0.53	6.5	--	2-4
1945	A	3000	0.53	6.5	--	2-4
1953-1973	AE	3000	--	6.0	4-7	2-4
1974-1977	AE	4500	0.455	6.5	5-7	1-2 1/2
1978-1988	A	4000	0.490	6.5	3-5	2-4
1978-1988	AE	4500	0.445	6.5	5-7	--
1978-1988	A	4000	0.490	6.5	3-5	--

A- non-air entrained concrete - AE air-entrained concrete

Gebhardt and Burrows

Gebhardt compared changes in cement chemical and strength properties between North American cements produced in 1953-54 to 1994. (Gebhardt R. F., December 1995), (Blaine, August 1965), (Clifton, 1971). Table 2 shows typical chemical and strength properties for cements being produced between the mid-1950's and mid-1990's. Type I and II are general purpose cements. Type III cement is high-early strength cement and Type IV has low heat of hydration properties. Modern (1994) general purpose cements, Type I or II, had chemical compositions similar to the Type III high-early strength cements produced in the 1950's. High C_3S usually results in rapid setting and high, early strength gain. Further the setting time and early age strength gain of the modern Type I or II cements is faster and much higher than even the Type III cements produced in the 1950's. Therefore, modern cement concretes would be expected to crack more due to their higher initial heat of hydration, higher early concrete modulus, and lower creep at early ages. Type IV low heat cement which would be expected to have a lower cracking tendency is unfortunately no longer produced in North America. The changes in the AASHTO concrete specifications and the evolution of the cement chemistry and early strength properties likely contributed to the increase in bridge deck cracking observed by DOT engineers in about the mid-1970's. (Krauss & Rogalla, Transverse Cracking in Newly Constructed Bridge Decks, 1996)

Table 2. Comparison of cement chemistry and strength properties for 1950's and 1990's cements

						Set (min.)		Compressive (psi)			
Type	C_3S	C_2S	C_3A	C_4AF	Blaine	Initial	Final	1 day	3 day	7 day	28 day
1994 (Gephardt)											
I	51.8	18.2	10.6	7.4	374	167	311	2485	3760	4620	5605
II	54.9	17.3	7.1	10.6	380	165	270	2463	3850	4786	5570
III	51.7	18.0	10.4	6.9	545	135	257	3981	5167	5923	6782
IV	42.2	31.7	3.7	15.1	339	189	315	900	1725	2529	5417
1950 (SP-127)											
I	44.6	27.4	11.2	8.3		280	440	520	1610	2760	4450
II	43.6	30.9	5.2	13.2		270	460	520	1400	2140	3790
III	52.9	18.6	10.5	9.5		195	360	1530	3680	5080	6340
IV	27.6	48.9	4.4	12.3		355	555	240	740	1220	2830

Burrows [1998] found that modern cements are more susceptible to cracking problems due to their high tricalcium silicate (C_3S), high fineness, and high alkali. The trends for concrete to develop strengths earlier, contain high cement contents, and have low water-cement ratio was believed to exacerbate cracking. To reduce the risk of cracking, Burrows recommended that cements should have a low C_3S content (less than 45 percent); be low alkali (less than 0.6 percent Na(eq)); and be a coarse grind (less than 320 sq. m/kg). Unfortunately, cement producers have not changed cement chemistry in an effort to reduce cracking and have stopped producing Type IV cement having low heat characteristics that could be helpful.

Altoubat and Lange

Calculating the stress needed to crack concrete decks in service is difficult. Altoubat and Lange [2001] found that predicting cracking based on the simple ratio of stress to strength at later ages was inadequate since the stress history, particularly at very early ages, was important. They concluded that large stresses at early ages likely produce permanent damage at the micro level that leads to cracking sooner than predicted by a simple stress-strength criterion. The ratio of the tensile stress to tensile strength at the time of cracking was estimated to be approximately 0.60 to 0.64 based on split tensile strength tests or 0.75 to 0.8 based on direct tensile strength tests. History dependence is also shown in creep behavior since the creep coefficient at any point in time includes contributions for the previous stresses and time steps. Creep coefficients increase rapidly at early ages due to high shrinkage stress development and they found that tensile creep about doubles the shrinkage strain required to cause cracking, irrespective of water to cement (w/c) ratio.

RESEARCH APPROACH

This research is focused specifically on providing practical and realistic recommendations to reduce bridge deck cracking in California. The scope of the investigation and research included the following:

1. Literature review of causes and potential solutions to early-age deck cracking
2. Review of measures taken by other DOTs to reduce early-age cracking
3. Review of Caltrans Specifications/Design Practice/Construction Policies
4. Field and laboratory testing of two newly constructed bridge decks

5. Analytical studies using finite element (FE) models and lattice modeling
6. Validation of potential solutions
7. Recommendations to Caltrans

The focus was on changes that can be implemented into current Caltrans design and construction specifications, and construction procedures. Previous studies and research combined with our field and laboratory testing and analytical studies that were conducted as part of this investigation are the basis for the recommendations. Academic and industry experts in research, construction, and design of bridges were surveyed to form a consensus that the recommendations are appropriate for implementation by Caltrans.

FIELD AND LABORATORY WORK

WJE completed instrumentation, monitoring, and concrete laboratory testing of two new concrete box-girder bridge decks in California: Markham Ravine Bridge in Lincoln and Olive Lane Bridge in Santee (northeast of San Diego). The objectives of the field and laboratory work were:

1. to gather information regarding concrete mixes currently used by Caltrans for the construction of new bridge decks,
2. to observe and document standard deck construction practices,
3. to observe and document current deck curing practices,
4. to instrument the decks and collect basic information identified in the literature that has an influence on shrinkage and deck cracking including: concrete temperature and internal relative humidity (RH) of cast concrete, ambient temperature and ambient RH, wind speed and direction, and concrete strain, and
5. to evaluate the occurrence of cracks during the first few hours after concrete placing and again a few months after the wet curing was completed.

The focus of this investigation was to determine what construction-related procedures may be adversely affecting deck cracking and to develop recommendations to prevent or reduce the risk of cracking.

During our investigation at the Olive Lane Bridge in Santee, WJE documented the installation of three different types of wet curing systems: burlap blankets, burlene blankets, and burlene plus insulation blankets. The objective of this part of our investigation was to assess the influence of the different types of curing blankets on the heat transfer between the concrete and the ambient air.

Concrete Mix

The mix designs for both bridges are provided in Table 3.

Table 3. Concrete Mix Design for Markham Ravine and Olive Lane bridges

Mix Design	Markham Ravine Bridge	Olive Lane Bridge
Cement, lb.	506	564
Fly Ash, lb.	169	188
Cementitious Material Content lb.	675	752
Coarse aggregate, lb.	1,858	1,155
Coarse aggregate size	1 in. maximum	1in. maximum
Fine aggregate, lb.	1,243	1,571
Water, lb.	284	330
Admixtures	Polyheed 1025, 27 oz	WRDA 64, 30.1 oz
		Daravair, 3.0 oz
Air content, percent	1.5	3.0

WJE performed early and later age laboratory testing of concrete samples cast during the construction of the two bridge decks. Laboratory tests reports for both bridges are included in Appendix A. Test results are summarized in Table 4. The mix design used for Markham Ravine Bridge has a lower total cementitious content (cmc), larger proportion of coarse aggregate, and a lower early age compressive strength and modulus of elasticity. The mix used for Olive Lane Bridge has a larger cmc, larger proportion of fine aggregate and higher early-age compressive strength and modulus of elasticity. While the laboratory-cured shrinkage results were similar between the mixes, the Markham Ravine Bridge concrete had a higher field-cured shrinkage values. Laboratory specimens were cured with water for 7 days while field specimens were left on the field for 7 days and subjected to curing similar to the deck.

Petrographic examination of the concrete indicated the concrete represented by a cylinder from the Markham Ravine Bridge contained a siliceous gravel coarse aggregate and a natural siliceous sand fine aggregate. The gravel had a nominal top size of 3/4 inch and both the coarse and fine aggregates appeared normal in gradation, distribution, and soundness. The cement and fly ash content totaled an estimated 7 to 7-1/2 bags per cubic yard. The fly ash content was in the range of 20 to 25 percent of the total cementitious materials. The w/cm was estimated in the range of 0.39 to 0.44. The sample was non-air-entrained and had an air content estimated at 2 to 2-1/2 percent. The cylinder was free of evidence of any distress mechanisms. The estimated mix proportions matched closely to the specified mix design.

Petrographic examination of a concrete cylinder from the Olive Lane Bridge indicated that the concrete was non-air-entrained and contained partially crushed siliceous gravel coarse aggregate, manufactured siliceous sand fine aggregate, an abundant total cementitious materials content estimated to be 7-1/2 to 8 bags per cubic yard, with 20 to 25 percent fly ash replacement, and a w/cm in the range of 0.40 to 0.45. The mix proportions were consistent with the provided mix design. No distress was detected in either concrete.

Table 4. Concrete Test Results for Markham Ravine and Olive Lane bridges

Laboratory Results	Markham Ravine Bridge	Olive Lane Bridge
Slump, in	4.0	5.0
Penetration, in	2.25	3.0
Plastic Unit Weight, pcf	153.2	144.3
Air content	1.7	2.6
Temperature of concrete, °F	67	75
Ambient temperature, °F	55	75
Compressive strength		
1 day, psi	880	2,540
2 days, psi	1,540	2,790
7 days, psi	3,090	3,700
14 days, psi	4,150	4,460
28 days, psi	5,180	5,290
90 days, psi	6,440	5,300
Splitting Tensile Strength		
1 day, psi	105	260
2 days, psi	170	215
7 days, psi	300	355
14 days, psi	350	345
28 days, psi	375	445
90 days, psi	476	540
Modulus of elasticity		
1 day, psi	1.35×10^6	2.62×10^6
2 days, psi	1.60×10^6	2.52×10^6
7 days, psi	3.10×10^6	3.20×10^6
14 days, psi	3.60×10^6	3.28×10^6
28 days, psi	3.50×10^6	3.58×10^6
90 days, psi	3.78×10^6	4.32×10^6
Shrinkage Field Specimen		
7 days, percent 14 days, percent 21 days, percent	0.008 -0.047 -0.055	0.002 -0.027 -0.038
28 days, percent	-0.065	-0.048
35 days, percent	-0.068	-0.052
Shrinkage Lab Specimen		
7 days, percent	0.002	0.003
14 days, percent	-0.020	-0.017
21 days, percent	-0.032	-0.023
28 days, percent	-0.038	-0.037
35 days, percent	-0.038	-0.047

Construction

WJE observed construction practices during deck casting of the Markham Ravine Bridge site in Lincoln, California on April 14, 2010 and during deck casting of the Olive Lane Undercrossing Bridge site in Santee, California on May, 27, 2010 (Figure 4 and Figure 5). Both bridges are posttensioned, box-girder type as shown in Figure 6 and Figure 7. Observations during construction included:

Markham Ravine Bridge

- Water was applied to the forms 30 minutes before concrete placement.
- Concrete placement began at 7 a.m. with air temperature of 55°F (13°C); wind speed of 5 mph, and relative humidity of 74 percent; and concluded around 12 p.m.
- Concrete was pumped and vibrated with a single internal (stinger) vibrator.
- Concrete temperature, RH within the concrete, and strains were recorded by WJE for 17 days.

Figure 4. Concrete placing for deck of Markham Ravine Bridge.

Olive Lane Bridge

- Water was applied to the forms 30 minutes before concrete placement.
- Concrete placement began at 7 a.m. with air temperature of 58°F (14°C); wind speed of 4 mph, and relative humidity of 81 percent and concluded around 3 p.m.
- Concrete was pumped and vibrated with a single internal (stinger) vibrator.
- Concrete temperature at placement was 75°F (24°C).
- Rebar temperature during placement was 79-100°F (26-38°C).

- Concrete temperature, RH within the concrete, and strains were recorded by WJE for 23 days.

Figure 5. Concrete placing for deck of Olive Lane Bridge.

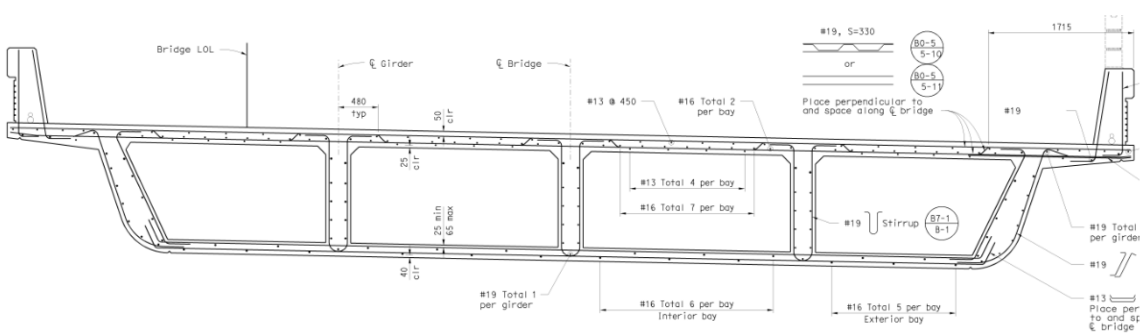


Figure 6. Typical cross section, Markham Ravine Bridge.

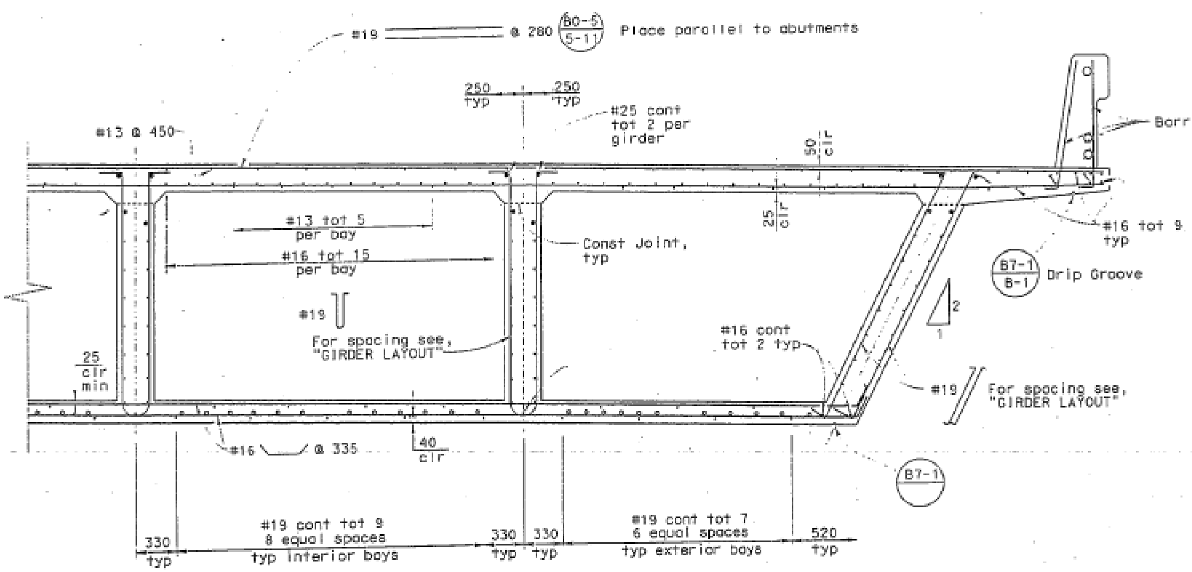


Figure 7. Typical cross section, Olive Lane Bridge.

Curing

Observations related to the curing practices during the casting of the Markham Ravine Bridge and Olive Lane Bridge decks include (Figure 8 through Figure 13):

Markham Ravine Bridge

- Curing compound was applied approximately 1 hour after concrete placement.
- Water curing started approximately 6 hours after concrete placement and was maintained for 7 days.
- Curing blankets were installed beginning 6 hours after concrete placing. Transguard 4000 (burlene) blankets were installed over the entire deck, including data stations A and B as described below.
- Plastic shrinkage cracks were observed typically 6 hours after placing started and before the blankets were installed. Some cracks appeared as soon as 2 hours after placing and in some instances before the curing compound was applied.
- Fogging was not performed.

Figure 8. Application of curing compound, Markham Ravine Bridge.

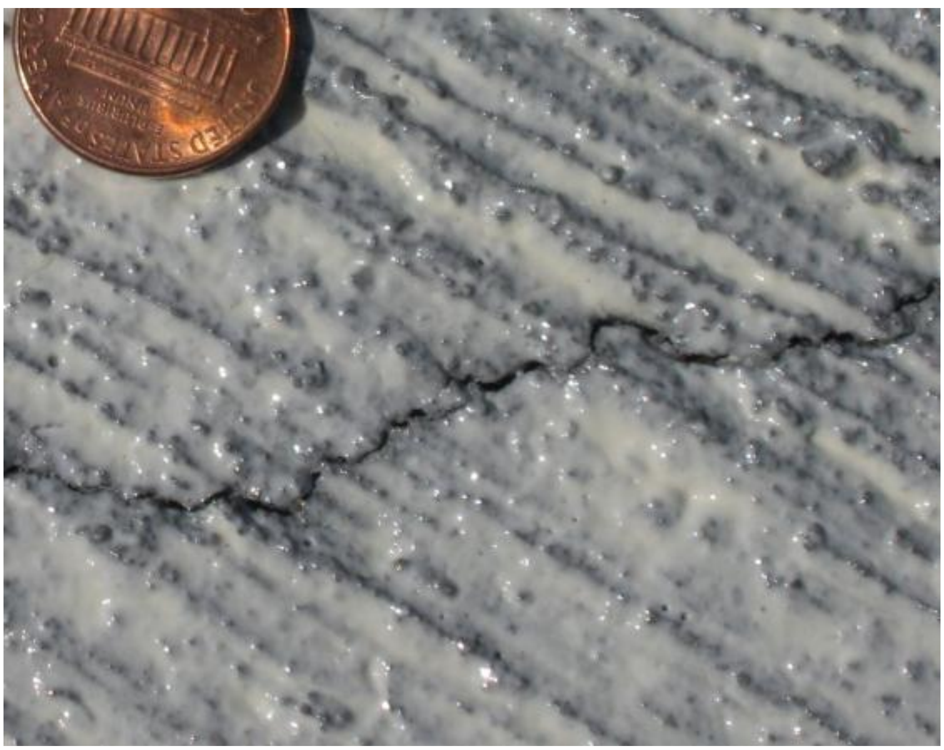


Figure 9. Cracks forming before the application of curing compound. Note liquid compound around crack edges. Markham Ravine Bridge.

Figure 10. Installation of curing blankets, Markham Ravine Bridge.

Olive Lane Bridge

- Curing compound application began approximately 2 hours after concrete placement.
- Plastic shrinkage cracks were observed within 2 hours of placement before curing compound was applied. Additional cracks formed within 6 hours after placement and before the blankets were installed.
- Curing blankets were installed 20 hours after concrete placement (next morning).
    - Transguard 4000 blankets (burlene) were installed at Zones B and C described below
    - Insulation blankets were installed in addition to burlene at Zone C
    - Two layers of burlap blankets were installed at Zone A
- Water curing started approximately 22 hours after concrete placement.
- Fogging was not performed.
- A second layer of curing compound was applied at Zone A after removal of curing blankets (eight days after concrete placing).

Figure 11. Application of curing compound. Olive Lane Bridge.

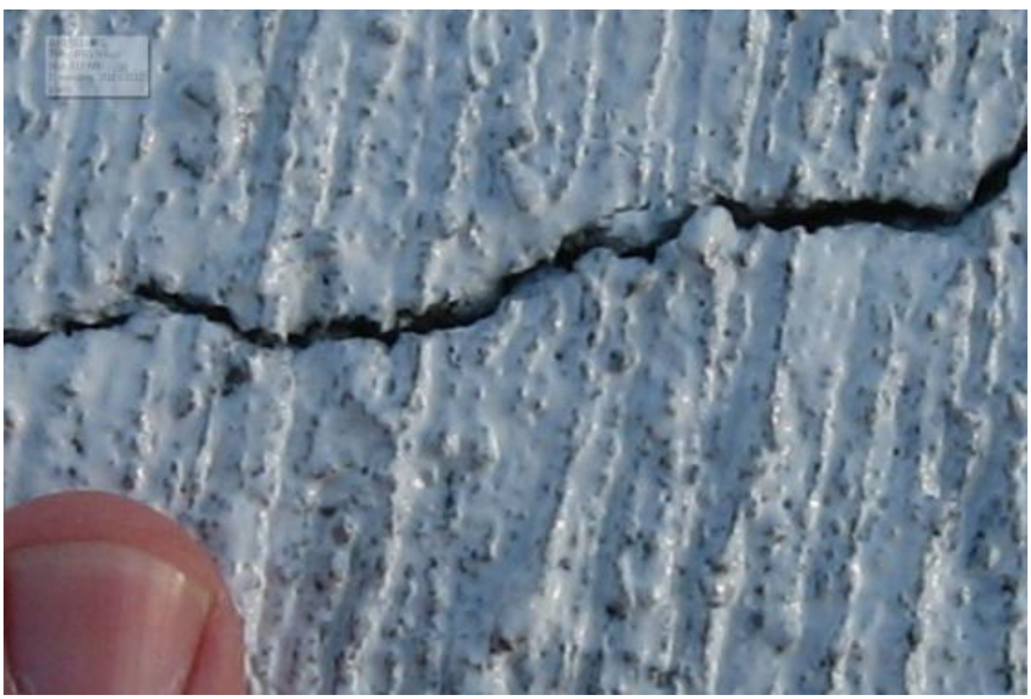


Figure 12. Cracks forming a few hours after concrete placement. Note compound within the crack. Olive Lane Bridge.

Figure 13. Installation of three types of curing blanket: burlap on the back (Zone A), burlene on the center (Zone B), and burlene plus insulation on the front (Zone C). Olive Lane Bridge.

Instrumentation

Markham Ravine Bridge

Prior to casting the deck, WJE staff, on April 12 and 13, 2010 installed instrumentation and data acquisition systems into the prepared bridge deck forms at the Markham Ravine Bridge site in Lincoln, California. On April, 14, 2010 the bridge deck was cast and sensor data acquisition began. Curing consisted of a single layer of curing compound and curing blankets as described above.

Shortly before the start of onsite work, WJE was told that the intended deck pour date would be April 16th. On arrival at the bridge site, WJE learned that the concrete pour date had been accelerated by two days; in addition, strong rainfall on April 12th prevented deck work. As a result, the planned four days for instrument and data acquisition system installation was reduced to one day. Working one sunrise to sunset day, WJE was able to install two sensor clusters. At the third point of the bridge from the west end, sensor clusters were installed at the mid-point between girders (Location A) and over the center girder (Location B). Figure 14 shows a diagram of the instrument cluster locations and the weather station installed on the Markham Bridge.

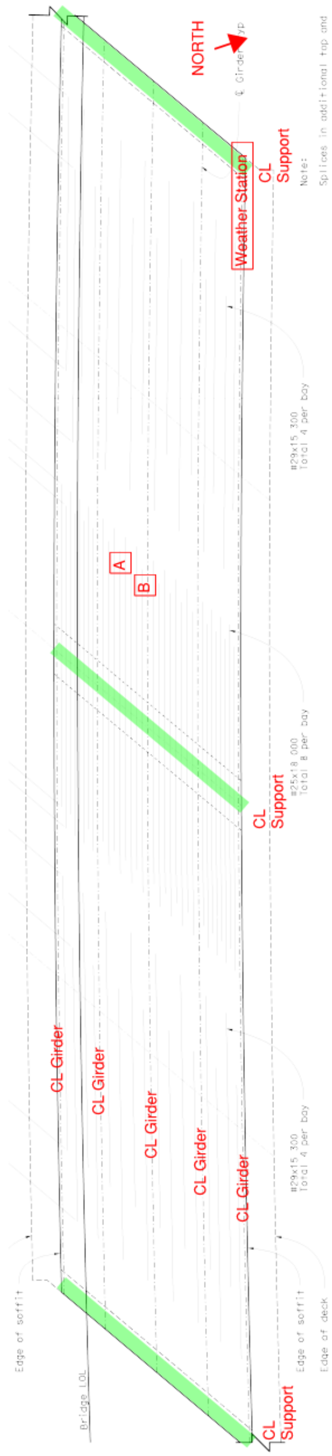


Figure 14. Instrument Location Designations at the Markham Ravine Bridge.

At Location A, strain gauges, thermocouples, and relative humidity gauges were located within the deck thickness as shown in Figure 15. For Location B, instruments placed over the girder were located at similar depths, but relative humidity gauges were not used. Three additional thermocouples were also placed on the bridge near Location B; two were drilled to depths of 1 inch and 18 inches (25mm and 457mm) in the girder and one was placed on the underside of the bridge. The weather station consisted of a wind speed recording device, a thermocouple for ambient air temperature, and a relative humidity gauge.

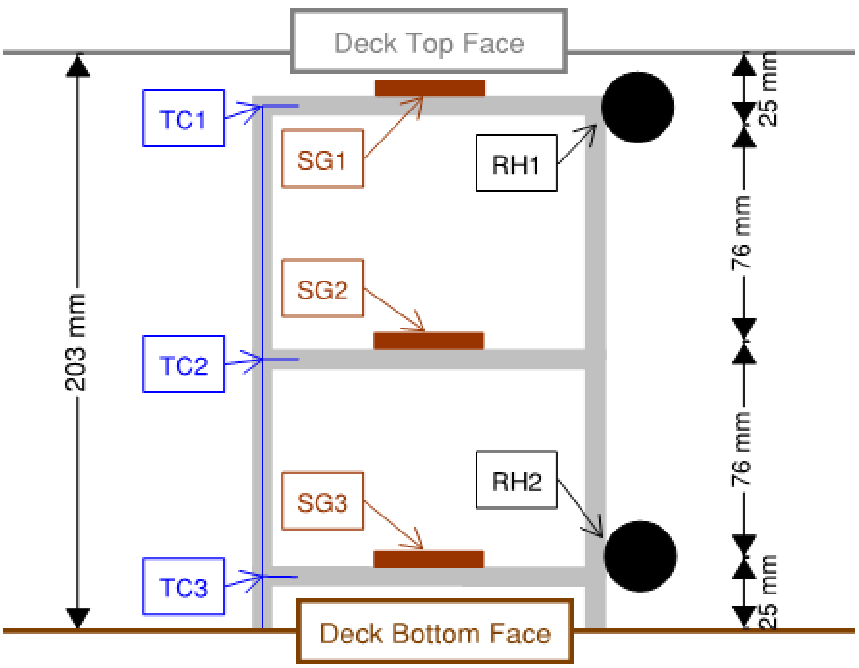


Figure 15. Typical Instrument Cluster Diagram at the Markham Ravine Bridge.

Olive Lane Bridge

WJE arrived on the Olive Lane Bridge site in Santee, California on May 23, 2010. From May 23 through May 26, 2010 instrumentation and data acquisition systems were installed into the bridge. On May, 27, 2010 half of the bridge deck was cast and sensor data acquisition was implemented throughout the deck placement process. Three different curing methods were used on the Olive Lane Bridge as described in the curing section above.

Figure 16 is a diagram of the instrument cluster locations, the curing type sections, and the weather station location. Four instrument clusters were placed in the bridge deck. Instrument clusters at Locations A, B, and C were placed at the mid-span between girders and at the mid-point of each the three curing method sections. The instrument cluster at Location D was placed over a girder at the mid-point of the standard curing method section. In addition to the in-deck instrument cluster at Location D, a strain gauge, thermocouple, and relative humidity gauge were placed on the underside of the bridge.

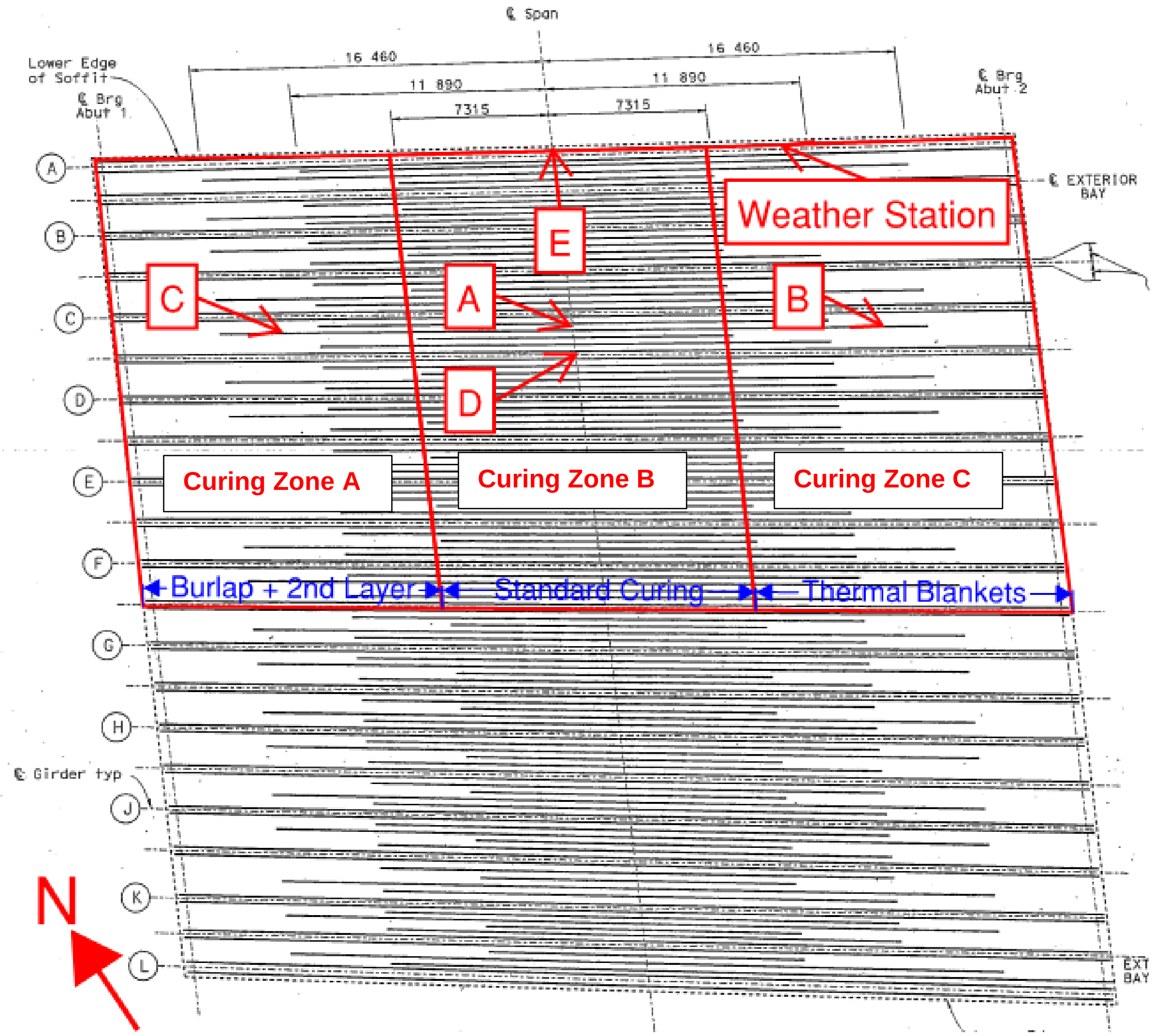


Figure 16. Weather Station and Instrument Locations, and Curing Method Areas on the Olive Lane Bridge.

At locations A, B, and C, strain gauges, thermocouples, and relative humidity gauges were located within the deck depth. For Location D, instruments placed over the girder were similarly located except relative humidity gauges were not used. Location D included a strain gage and a thermocouple attached to the exterior of the edge girder. A weather monitoring station was placed on the north edge of the deck near the east end. The weather station consisted of a wind speed recording device, a thermocouple to monitor ambient air temperature, and a relative humidity gauge. Figure 17 shows a diagram of the instrument types and locations for a typical cluster.

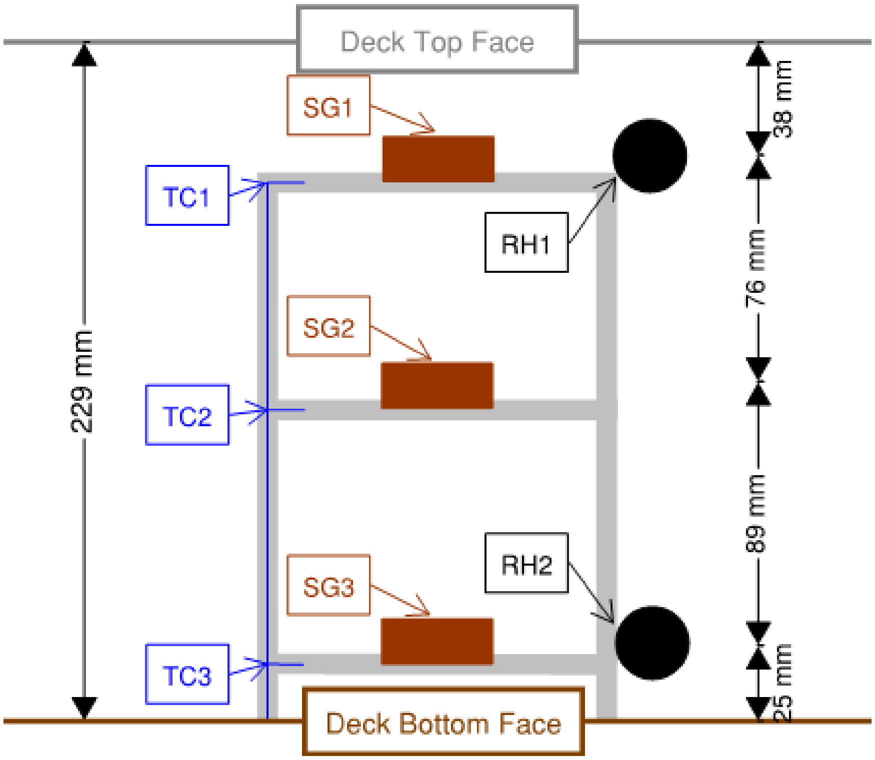


Figure 17. Typical Instrument Cluster Diagram at the Olive Lane Bridge, Santee, CA

General Instrumentation

Instruments were installed at three depths within the decks to capture concrete properties (strain, temperature and relative humidity) through the depth of the decks. Maintaining intended instrument alignment during concrete placement operations, providing support for instruments at different depths, and the variability of reinforcing alignment were necessary considerations when designing and installing the instrument clusters. The solution consisted of onsite fabricated aluminum cages that were assembled into the deck reinforcing steel and then guy-wired into place. Figure 18 shows an aluminum support cage assembled into the reinforcing steel before being fitted with instruments and guyed into place.

Figure 18. Instrument Cluster Support Structure, Markham Ravine Bridge.

Figure 19 shows a photo with the three monitoring instrument types labeled. The aluminum support structure was sized and drilled to securely mount each instrument at the specified depth. In order to allow the strain gauges to move axially while maintaining alignment, the notched end of the gauge (Figure 20) was fixed in place and the opposite end prevented lateral movement while leaving the axial direction free to move.

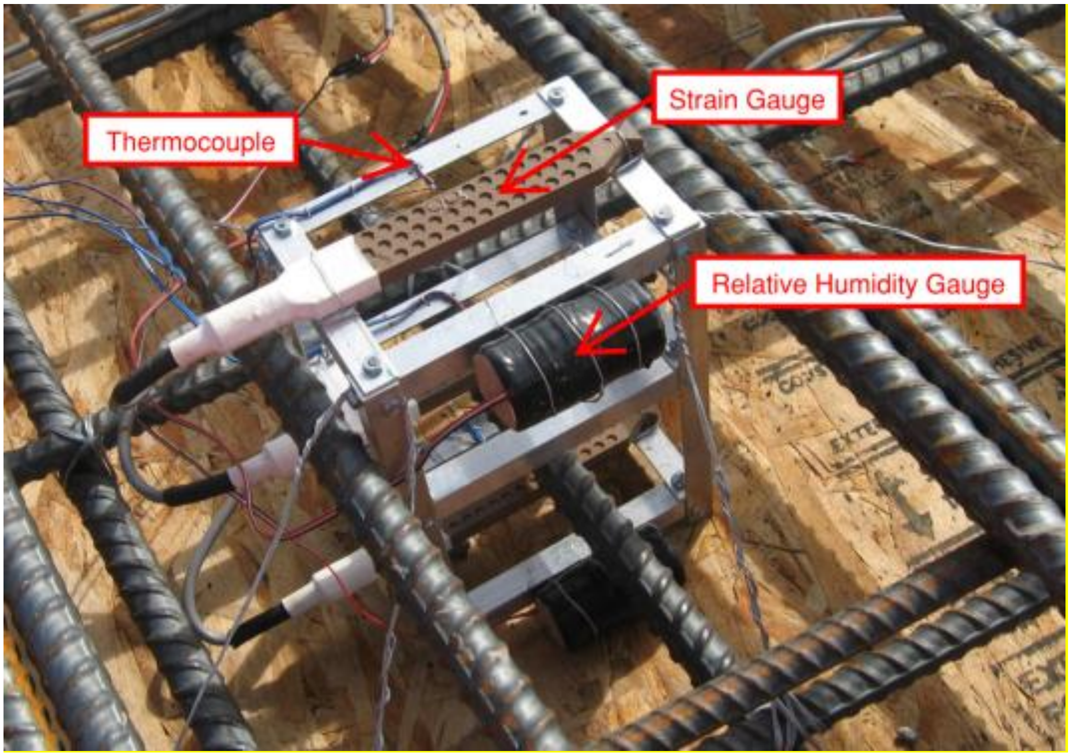


Figure 19. Instrument Types in a Typical Instrument Cluster, Markham Ravine Bridge.

The instruments were connected to their respective data acquisition wires. The data wires were strain relieved near the instrument clusters to prevent accidental wire failure during concrete casting. The wires were then run through the deck and located between the reinforcing layers, and routed to the edge of the bridge deck and into the data acquisition system. After the instruments were fitted, the support system was guyed into place and readied to be cast into the concrete deck. Figure 20 shows an assembled, instrumented, and supported cluster.

Figure 20. Typical Installed Instrument Cluster.

Throughout concrete casting operations WJE personnel were onsite to prevent instrument damage from construction work and to resolve any disruptions the instrumentation project could present to the construction operations; in addition, a WJE technician remained to monitor and ensure that data acquisition was being gathered prior to, during, and after the instruments submersion in concrete. Figure 21, shows a typical instrument cluster during concrete pouring and vibrating operations.

Figure 21. Typical Instrument Cluster during Concrete Pouring Photos.

The data acquisition system used battery power at the Markham Bridge and grid power with battery backup at the Olive Lane Bridge. Individual instrument wires were connected to the data collection system and run through a wireless router to allow for remote data monitoring. The system was contained in locked, heavy duty, steel gage box to prevent tampering and theft. Figure 22 is a photo of the finished data acquisition system.

Figure 22. Data Acquisition System Photo.

Data Acquisition

Acquisition of instrument readings began prior to, and continued through concrete placement and curing operations. Appendix B contains the collected data. For the Markham Ravine Bridge, instrument readings were continued for 17 days when stripping of the concrete forms and shoring necessitated removal of the data acquisition system. At the Olive Lane Bridge, data recordings were acquired over 23 days. Based on a review of the data collected, all instruments appear to have functioned properly.

Figure 23 shows the recorded temperature from the Location B gauge cluster over a girder in the Markham Bridge. The embedded thermocouples located at three depths within the deck depth capture the temperature variation throughout the section. As would be expected, the top of the deck is shown to have both higher variation between daily recorded highs and lows, as well as more rapid temperature changes. Conversely, the temperature recordings from the sensor 7-inches (178 mm)down from the surface show the expected lower daily variations and delayed temperature change with respect to the upper sensors. At times the lower sensors show temperature readings higher than the upper sensors. This is an expected result as the interaction of the convective, conductive, and radiative heat transfer in this location directly over a girder retains thermal gains longer than at other measured locations. Top deck temperatures and the temperature fluctuations tended to increase after the wet curing media was removed at about 8 days.

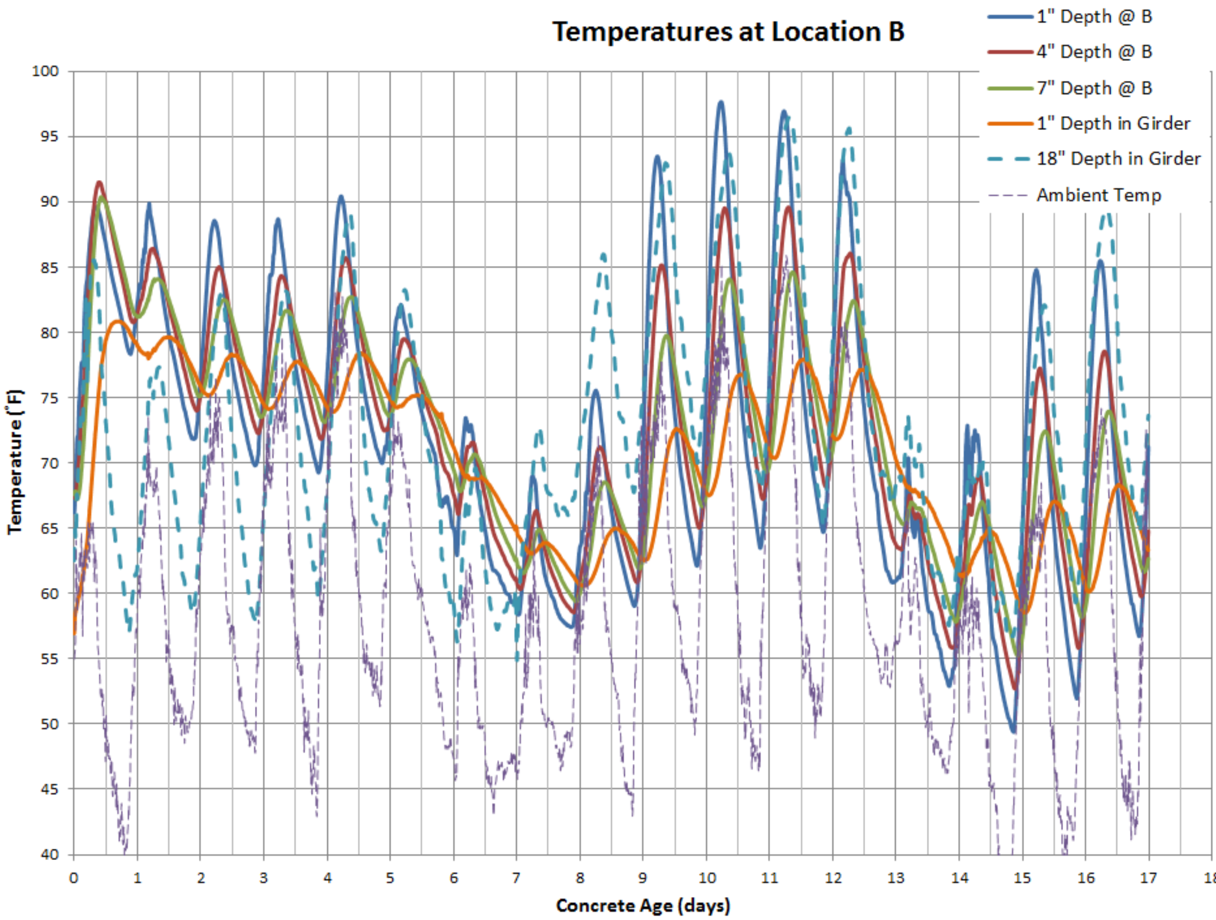


Figure 23. Temperature variation through deck thickness of Markham Ravine Bridge.

The variation in recorded strain at Locations A and B gauge clusters in the Markham Bridge are illustrated in Figure 24. The temperature variations shown in Figure 23 resulted in thermally induced cyclical material strains as shown in Figure 24 were increments in strain values mean expansion and decreases mean shrinkage. Strains and strain fluctuations increased similarly to temperatures after the wet curing blankets were removed at about 8 days. In addition, just before 12 days, a drop in tensile strain was observed because the bridge was post-tensioned. The resulting compression of the deck section due to the post tensioning force reduces the non-cyclical tensile strains due to shortening. Although data collection ends shortly thereafter, the expected reduction in non-cyclical tensile strain is maintained and the daily cycling due to temperature variations continues.

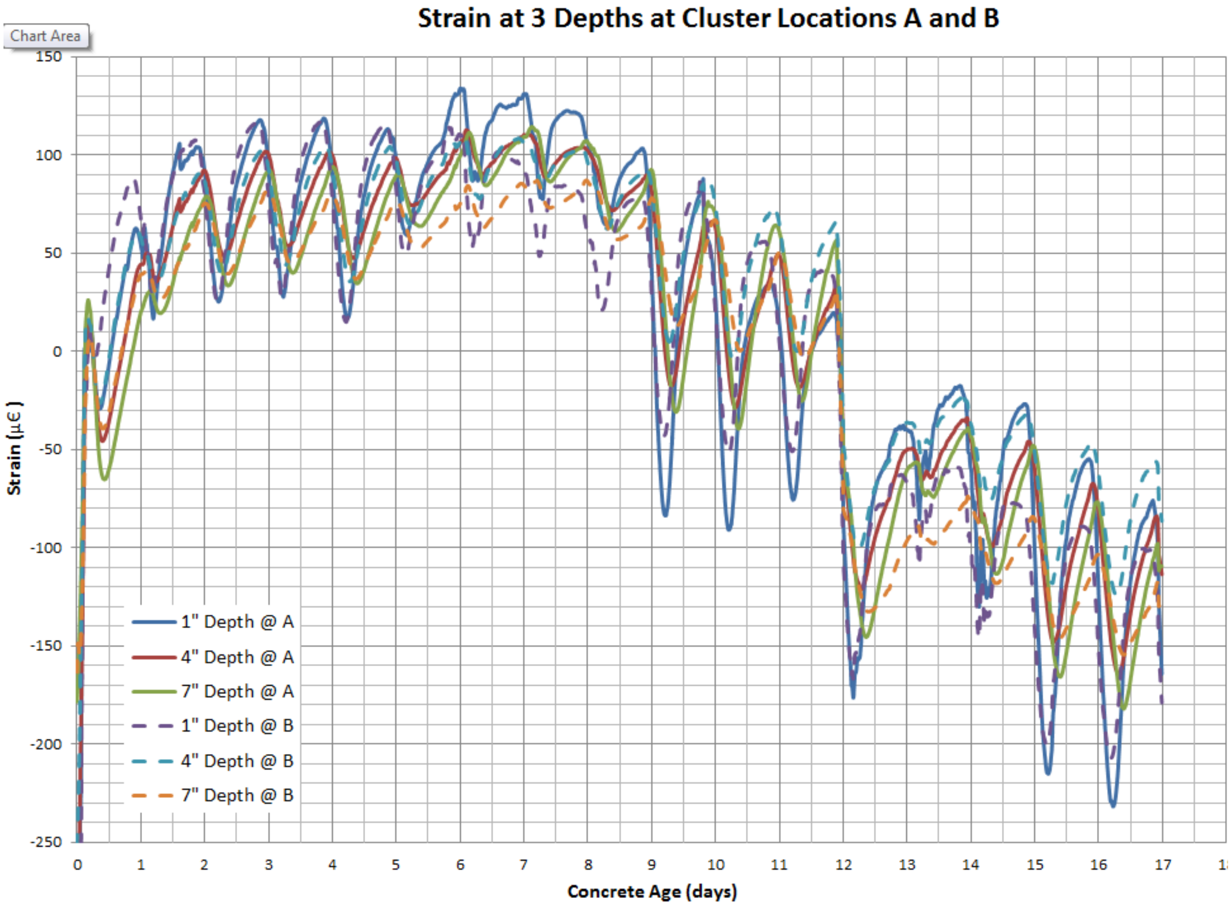


Figure 24. Tensile strain through deck thickness of Markham Ravine Bridge. Note the influence of prestressing at day 12.

Figure 25 shows the temperature and strain evolution over the first 14 days after concrete placement at the Olive Lane Bridge. It is clear that the installation of thermal blankets helps to reduce the thermal variations due to daily thermal cycles and maintained the concrete at a higher temperature. Note that in areas of the deck with thermal insulation, daily temperature oscillations average 6 to 8°F (3°C) while areas of the deck with no insulation average daily oscillations of 10 to 12°F (6°C). Once the thermal blankets were removed (approximately at day 8), the temperatures at the three different locations are very similar. Interestingly, strain in the concrete with insulation moved opposite to the other two sections and strain variations are also reduced for the deck areas with thermal insulation blankets until the curing blankets were removed. The strain at the gage location was near zero at eight days when the blankets were removed. Thermal blankets were installed 20 to 24 hours after concrete placing and appeared to have an effect that could be beneficial to reducing deck cracking. However, if thermal blankets are installed before the peak hydration heat; they may increase the maximum peak temperature of the concrete and potentially increasing stresses that could cause cracking. Research in to the optimum time to install blankets is suggested.

The application of the second coat of curing compound was applied after the wet curing was removed at Zone A (standard curing - burlap) and reduced the maximum diurnal surface temperature of the concrete by about 4°F (2°C), as shown in Figure 25. This also had a slight effect at reducing deck strains. Further it would also be expected to reduce the overall rate of drying and shrinkage during the first few months slightly.

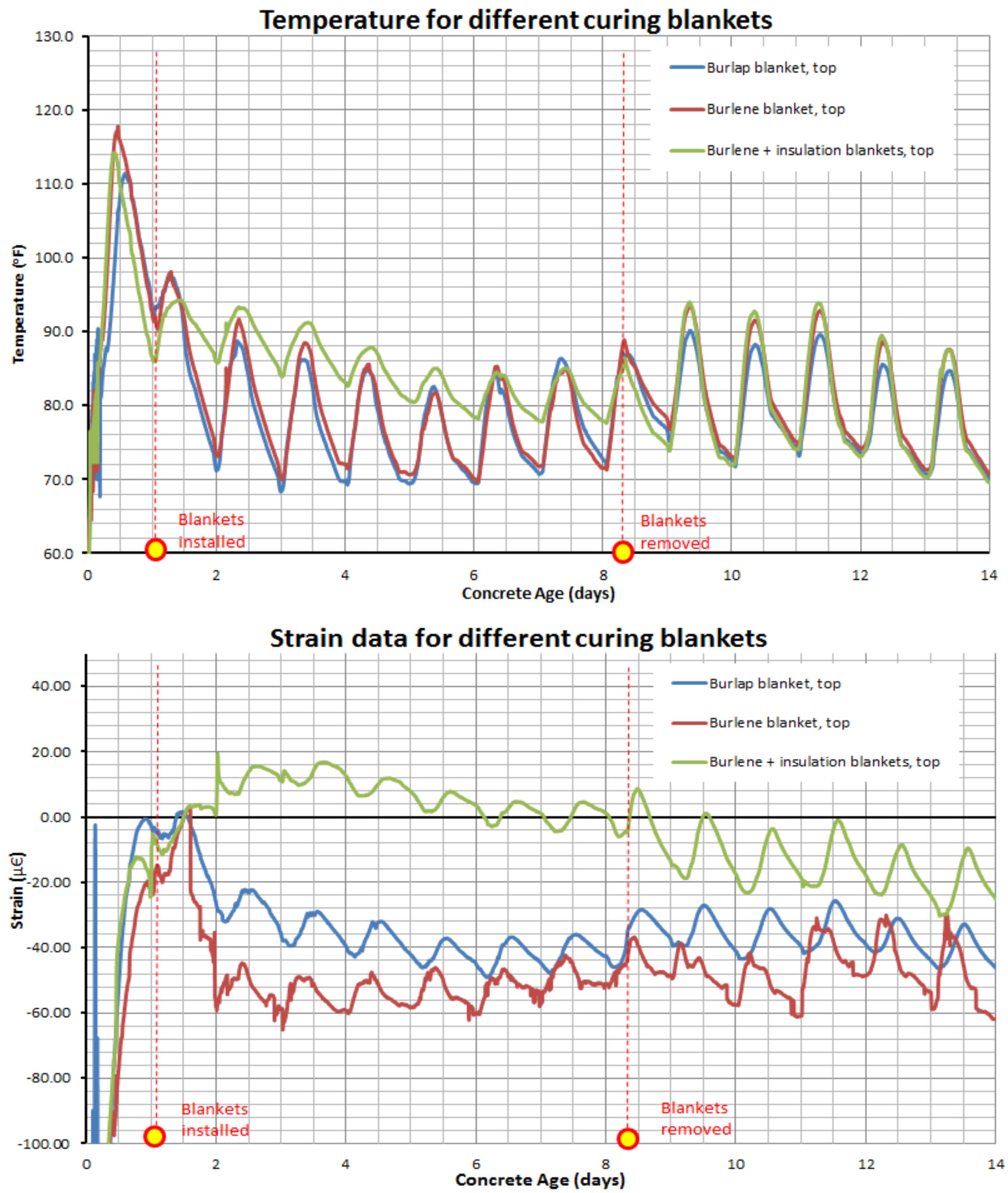


Figure 25. Temperature and relative strain measurements at 1 inch from top of the deck and three different curing blankets. Olive Lane Bridge.

Bridge Deck Cracking Surveys

WJE examined three times the Markham Ravine bridge deck for cracking, and two times the Olive Lane bridge deck. The first deck observation and documentation occurred after application of curing compound but within 6 to 8 hours of the concrete placement for both bridges. A follow-up inspection was performed at 21 days after casting the Markham Bridge. Finally, another crack survey was performed on August 8, 2010 for both bridges, approximately 16 weeks after construction of the Markham Ravine Bridge deck, and 10 weeks after bridge deck construction of the Olive Lane Bridge.

Crack Survey Within 6-8 hour After Concrete Placement

At both bridges, plastic shrinkage cracks were visible within 6 to 8 hours of concrete placement. Plastic shrinkage cracks typically occur on freshly placed concrete shortly after finishing operations but before the concrete has reached final set. In both bridges, they manifested as shallow surface cracks typically 0.015-0.019 inches (0.4-0.5mm) wide and from 11 to 33 inches (300-900mm) in length. Commonly they result from high surface evaporation rates occurring from high concrete surface temperatures and or high winds. The cracks were typically parallel to each other but not in-line with reinforcing. On the Markham Bridge the plastic cracks size (0.015-0.019 inches or 0.4-0.5mm) and distribution (approximately spaced at 33 inches or 900mm) were fairly consistent over the deck area as shown in Figure 26 and Figure 27. The Olive Lane Bridge showed some areas with significantly higher crack densities (approximately spaced at 12 in. [300mm]) and the cracks in high density areas were typically wider (0.02 to 0.03 inches or 0.5-0.75mm). The southwest end of the bridge had the highest density and width among the cracks recorded on the deck; not coincidentally, the southwest end of the bridge experienced direct wind impact on the day of casting. Cracking density on the remaining portion of the deck decreased when moving along the span away from the southwest end and remained fairly consistent with spacing at approximately 45 inches (1.2m). Figure 28 shows survey photos of plastic shrinkage cracks on the Olive Lane Bridge deck.

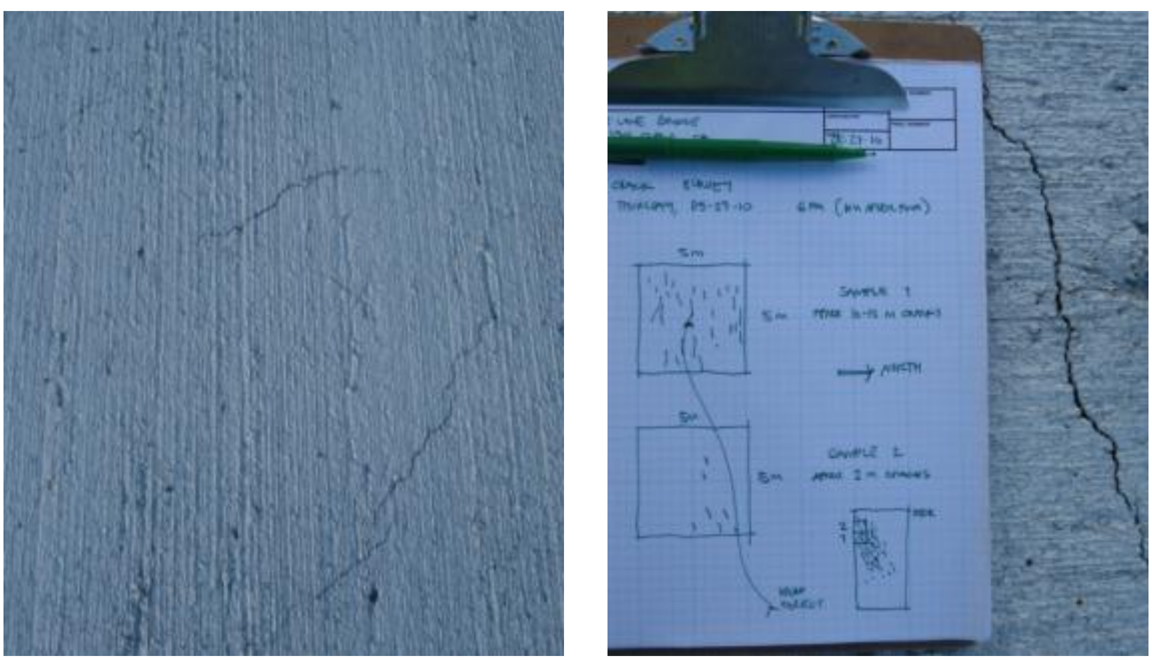


Figure 26. Within 24 hour of concrete placement - deck crack survey photos.

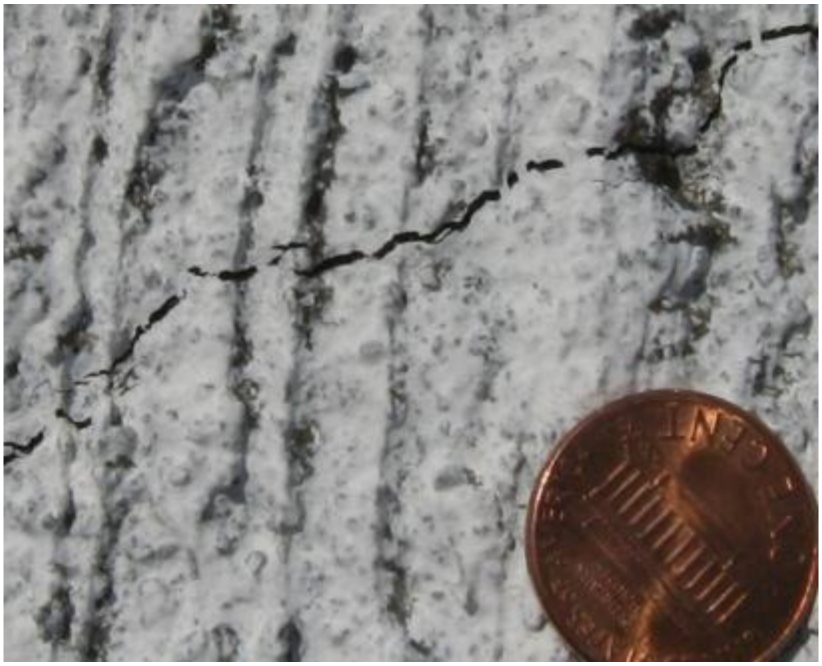


Figure 27. Plastic shrinkage crack after curing compound was applied. Note curing compound spanning over crack at some points. Markham Ravine Bridge.

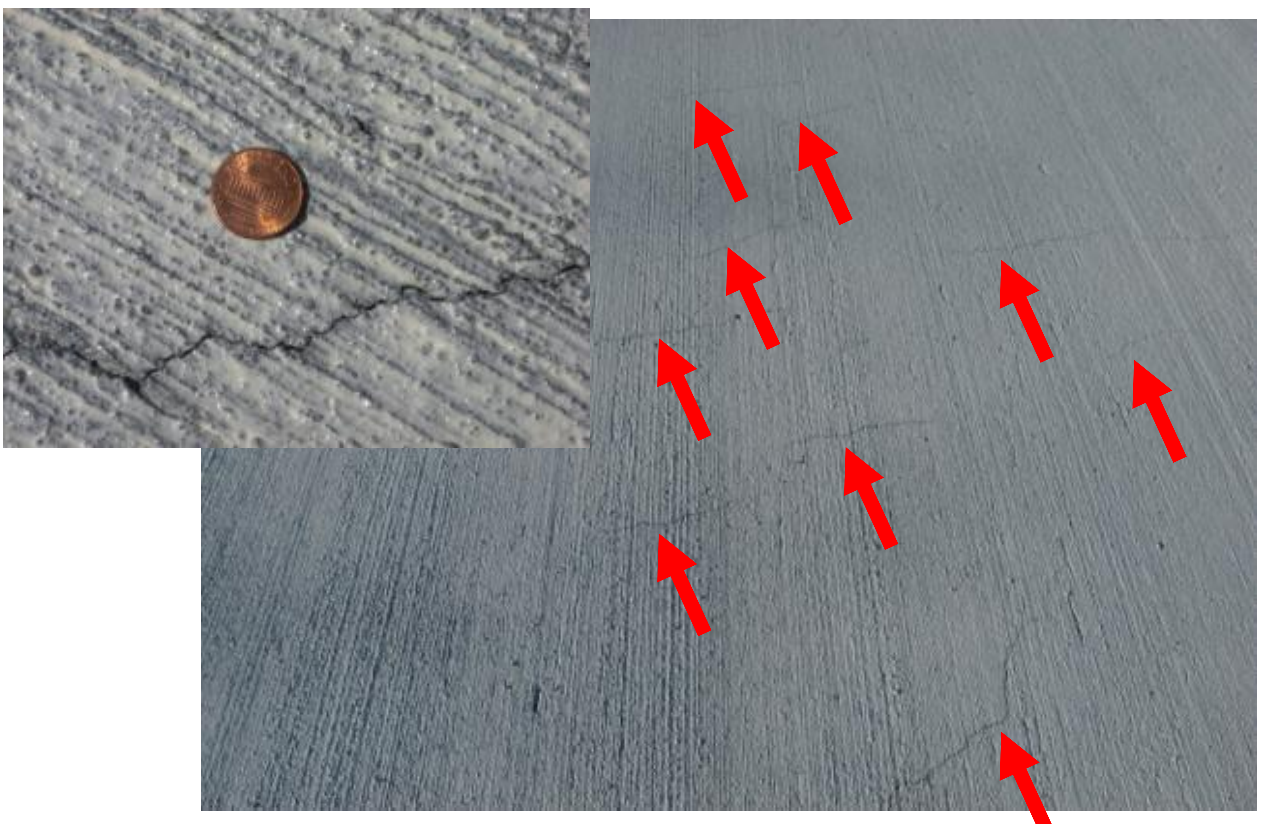


Figure 28. Plastic cracking observed 6 hours after concrete placing, Olive Lane Bridge.

Crack Survey After Water Curing Was Completed

A second crack survey took place at the Markham Ravine Bridge 21 days after the concrete placement to identify early-age bridge deck cracking. This was after the deck had been post-tensioned and about 7 days after the moisture curing was removed. The deck had not been open to traffic. Early-age cracks are typically oriented perpendicular (transverse) to the span and often run through the deck section depth. Identification of early-age cracks was difficult for two main reasons. One was that the deck concrete was finished with grooved texture lines. These lines are oriented perpendicular to the span; which is the same orientation that early-age cracks would be expected to form. The grooves also collect dirt and debris which further complicated crack identification. In the case of the Markham Bridge, a layer of sand was present over the entire deck surface. Another obstacle to identifying early-age cracks was due to the application of post tensioning. Axial compression from the post tensioning force has the effect of closing cracks oriented perpendicular to the deck span, such as early-age bridge cracks. Figure 7 shows photos from the second survey of the Markham Bridge deck.

The pictures in Figure 29 show a strip of survey area where the sand was removed; the crack photos in the figure show the plastic shrinkage cracks, however, early-age transverse cracks were not positively identified at this time.

Figure 29. Three weeks after concrete pouring deck cracking survey photos, Markham Ravine Bridge.

Final Crack Survey

A third crack survey took place approximately 16 weeks after the concrete placing of Markham Ravine Bridge, and 10 weeks after concrete placing for Olive Lane Bridge. For the Markham Ravine Bridge, we used an industrial power washer to clean a section of the deck and facilitate the identification of cracks, see Figure 30. We were able to observe both transverse and longitudinal cracks as depicted in the crack survey maps in Appendix B (Figure 33). Transverse cracks were mostly fine with surface widths ranging from 0.004-0.016 inches (0.1 to 0.4mm). For the Olive Lane Bridge survey, cleaning efforts were limited to brooming and applying water with a sprayer. Transverse cracking was not identified with minor exceptions. Refer to Appendix B for a crack survey figure.

Figure 30. Crack survey after power washing a section of the deck. Markham Ravine Bridge.

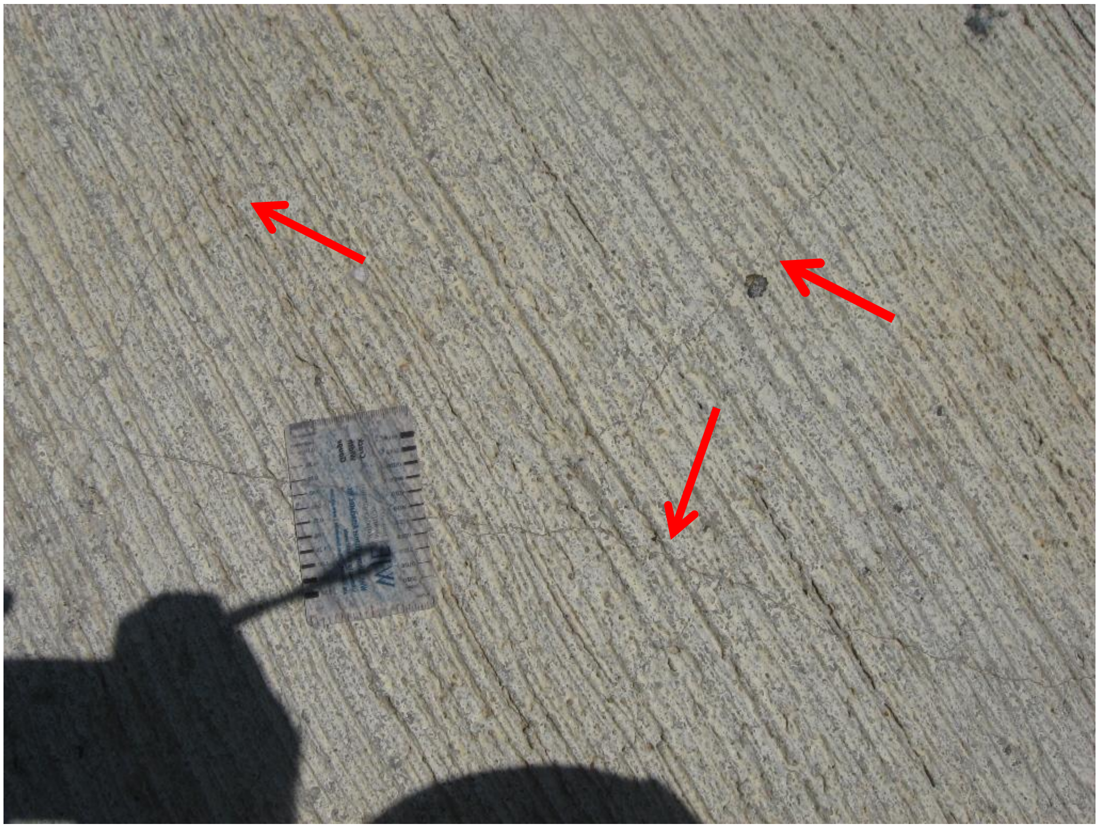


Figure 31. Longitudinal (upper) and transverse (lower) cracks observed during final survey at Markham Ravine Bridge.

Findings from Field and Laboratory Work

- Caltrans specification limits the cementitious material content of deck slabs to 675-800 lb/CY. The concrete mix used in both bridges met the specification limits.
- Concrete strength met the design strength at both bridges.
- Air content was observed to be within Caltrans specified values.
- Caltrans limits the amount of free water in the mix to 310 lbs/CY plus 20 pounds for each required 100 pounds of cementitious material in excess of 550 pounds per cubic yard. Both mixes complied with these limits.
- Caltrans specification limits the free shrinkage of deck concrete to 0.045% (450 microstrain) at 28- days. The concrete from the two bridges evaluated as part of this study had 28-day shrinkage values of 0.065 and 0.048 percent for field cured samples and 0.038 and 0.037 percent for laboratory cured samples. Shrinkage reducing admixtures (SRAs) were not used. The curing procedure for the specimens is described in Appendix A.
- The early curing methods used on the two bridge decks monitored for this study generally followed Caltrans specifications and standard practices but did not prevent widespread plastic shrinkage cracks.
- The average first-day concrete temperature is roughly 20°F (10°C) warmer in the Olive Lane Bridge compared to the Markham Bridge. This may be a result of the higher cementitious content in the concrete used for the Olive Lane Bridge.

Transverse cracking was only observed at the Markham Ravine Bridge which has two spans and an intermediate support. The transverse cracks were more frequent near the intermediate support. Transverse cracking was not observed at the Olive Lane Bridge.

- Thermal blankets installed after the peak temperature due to heat of hydration reduced the temperature changes and strains in the deck and kept the deck warmer.
- Application of a second coat of curing compound after wet curing was complete reduced peak diurnal temperatures by about 4°F (2°C) compared to the single coat applied after finishing but before wet curing.
- Data collected from the field work can be used in analytical modeling to predict cracking stresses.

General Conclusions:

- Improve curing procedures are needed to prevent plastic shrinkage cracking. Apply pre-saturated burlap or cotton-matting wet curing media immediately after strike-off and finishing (cotton mats without plastic backing can be placed dry and then thoroughly wetted). Mist the concrete and curing media to keep surfaces moist. Follow with second layer of wet curing media. Ensure adequate equipment is available to lift and place heavy pre-wetted burlap without damage to the surface. Consider screed mounted rolls of burlap. Keep curing media saturated. Cover with soaker hoses and white plastic no later than twelve hours after concrete placement. This procedure can affect deck tining so grooving is normally done after the concrete is fully hardened. Curing compounds were not found to be effective before wet curing and are not needed if misting and immediate wet curing is applied. Applying curing compound after the wet curing is complete is recommended to reduce peak diurnal temperature and drying stresses slightly.
- Review Caltrans Bridge Construction Records and Procedures Manual (Memo 105-4.0) on preventing plastic shrinkage cracking with the Contractor and project staff prior to deck placement.
- Require that fogging equipment be available to reduce high evaporation rates.

ANALYTICAL STUDIES OF CALTRANS DECKS

Parameter Study of Bridge Geometry and Material Properties for Uniform and Linear Free Deck Strains

WJE studied the effects of various box girder geometries for several (simplified) linear temperature and shrinkage differences in the deck or between the deck and webs and bottom soffit. Table 5 summarizes the range of box girder geometries considered.

Table 5. Box Girder Geometries

	Overall Height	Deck Thickness	Web Thickness	Bottom Soffit Thickness	Web Spacing
Minimum:	1000 mm (39.4 in.)	150 mm (5.9 in.)	300 mm (11.8 in.)	150 mm (5.9 in.)	2000 mm (78.7 in.)
Maximum:	2500 mm (98.4 in.)	250 mm (9.8 in.)	300 mm (11.8 in.)	250 mm (9.8 in.)	4000 mm (157 in.)

For all concrete, we assumed a coefficient of thermal expansion of $5.5x10^{-6}/°F$ ($9.9x10^{-6}/°C$) (often considered a common value), and an effective Poisson ratio of 0.15 (about half of the typical value, to account for only limited lateral restraint of the deck and corresponding effect on longitudinal stresses). To account for various concrete age and corresponding creep potential, an effective elastic modulus (Eeff) was estimated for the various temperature and shrinkage combinations.

A system of equations that assumed linear elastic behavior was developed, assuming stresses were uniform across the width for any specific height. Two three-dimensional finite element analysis (FEA) models were created to check the general accuracy of the equations, and the stresses calculated by the equations were typically within 10 percent of those calculated by the FEA models. Table 6 summarizes the five temperature and shrinkage combinations considered for the parameter study, along with the maximum and minimum calculated tensile stresses in the bottom of the deck (where tensile deck stresses are largest) and corresponding geometry. Free strains were only applied to the deck, to model either free strain differences within the deck or free strain difference between the deck and underlying webs and bottom soffit. For all studies, the free strains (temperature change or shrinkage) were based upon expected average value for concrete decks in California; because these analyses assumed elastic behavior, the calculated stresses are directly proportional to the applied free strains (e.g., if the applied free strain is 20% larger, the calculated stresses would also be 20% larger for all other variables being the same). For all cases, the maximum deck tension occurred at the bottom surface of the deck.

Table 6. Parameter Study Temperature and Shrinkage Combinations

Value	**Parameter Study 1**	**Parameter Study 2**	**Parameter Study 3**	**Parameter Study 4**	**Parameter Study 5**
Applied free strain, deck top	-40°F temperature (early cooling)	-10°F temperature (early cooling)	none	-500 microstrain (shorter-term shrinkage)	-700 microstrain (longer-term shrinkage)
Applied free strain, deck bottom	-40°F temperature (early cooling)	none	-10°F temperature (early cooling)	-500 microstrain (shorter-term shrinkage)	-700 microstrain (longer-term shrinkage)
Assumed E_{eff}, deck	540 ksi	1,500 ksi	1,500 ksi	1,104 ksi	1,104 ksi
Assumed E_{eff}, webs and soffit	4,414 ksi	4,414 ksi	4,414 ksi	1,472 ksi (newer web with high creep)	2,208 ksi (old web with lower creep)
Calculated maximum deck tension, worst case, and…	116 psi	-17 psi (compression)	80 psi	298 psi	505 psi
… corresponding geometry	maximum height; minimum deck thickness, web spacing, and soffit thickness				
Calculated maximum deck tension, best case, and…	51 psi	-38 psi (compression)	62 psi	123 psi	200 psi
… corresponding geometry	minimum height; maximum deck thickness, web spacing, and soffit thickness		minimum height; maximum deck thickness, web spacing; moderate soffit thickness	minimum height, soffit thickness; maximum web spacing; moderate deck thickness	

Because the analyses assumed linear elastic behavior, the calculated stresses for a given applied free-strain magnitude can be simply multiplied by a loading factor to estimate the stresses for a scaled loading. For example, if the temperature drop modeled in Study 1 was 60°F (16°C) instead of 40°F (4°C), a 50 percent increase in temperature drop, the calculated stresses would be 50-percent larger.

The parameter study is not intended to predict stresses accurately for a specific condition, but rather to examine approximate stresses and the relative effects of geometry on deck stresses. Maximum tensile stresses in the deck always occurred on the bottom surface of the deck. From the parameter study, it was clear that deck stresses were always largest when stiffness of the deck was smallest and the stiffness of the web and soffit (that provided the deck restraint) were largest. In other words, deck stresses are largest, and the cracking tendency is greatest, when the deck is thinnest, the overall box section is deepest, and the web spacing is narrowest. The thickness or properties of the bottom flange soffit (underside of superstructure) typically influenced deck stresses only a very small and negligible amount.

Of all the geometry factors examined, the overall height of the box girder had the largest influence on deck stresses. However, because the span length largely determines the required girder depth, modifying the girder depth to reduce the risk of deck cracking will typically not be feasible. Reducing the span lengths, when feasible, and correspondingly reducing the box girder depths, can substantially reduce the risk of transverse deck cracking.

The deck thickness did not have a large effect on deck stresses when the concrete was very young (when the concrete had a small modulus of elasticity and high creep potential), but it had a moderate effect when long-term shrinkage was considered (increasing the deck thickness from 6 to 7 inches (152-178mm) decreased shrinkage stresses in the deck roughly 10 percent). Similarly, increasing the web spacing did not substantially affect stresses when the concrete was very young, but increasing the spacing from 79 to 98 inches (2-2.5m) often decreased deck stresses by almost 10 percent. To reduce the risk of deck cracking at least a small amount, web spacing can be increased and deck thickness can be increased; structurally, increasing the web spacing will typically require a thicker deck or deeper or wider web, with the costs of the change somewhat offsetting each other.

Parameter Study of Early Temperatures and Stresses

The software 4CTemp&Stress[1] was used to study the early temperatures in new concrete decks on concrete box bridges, for various geometries and environmental conditions.

Geometry

Of the concrete box bridge drawings Caltrans provided to us, the Coon Creek Right bridge had the smallest box section and would provide the least deck restraint, and the Olive Lane Undercrossing was the largest and would provide the largest deck restraint. These two bridges were selected for the parameter study of early temperatures and stresses. Figure 32 shows the modeled cross-section of the Coon Creek Right Bridge, and Figure 33 shows the modeled cross-section of the Olive Lane Undercrossing Bridge. Both sections included one web of the bridge, and its tributary width of deck and soffit (that extended midway to the adjacent webs). Because the I-shaped sections were symmetric about a vertical axis, only a half was modeled to speed analysis time without affecting results (Figure 34).

[1] 4C-Temperature&Stress version 2.10, rel. 7, Danish Technological Institute.

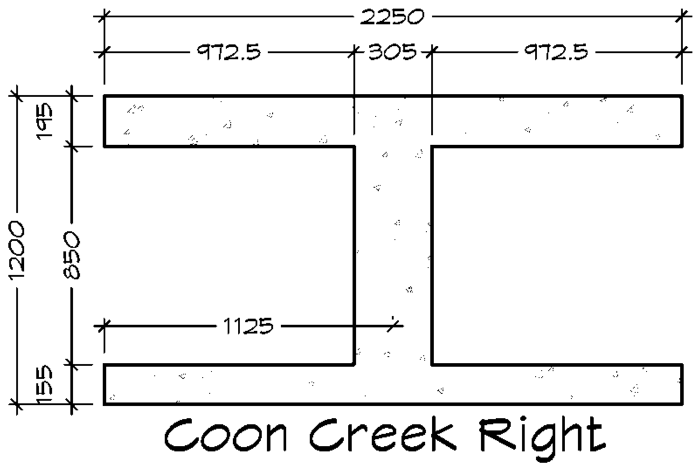


Figure 32. Modeled cross-section for Coon Creek Right bridge (smallest box section; dimensions in mm)

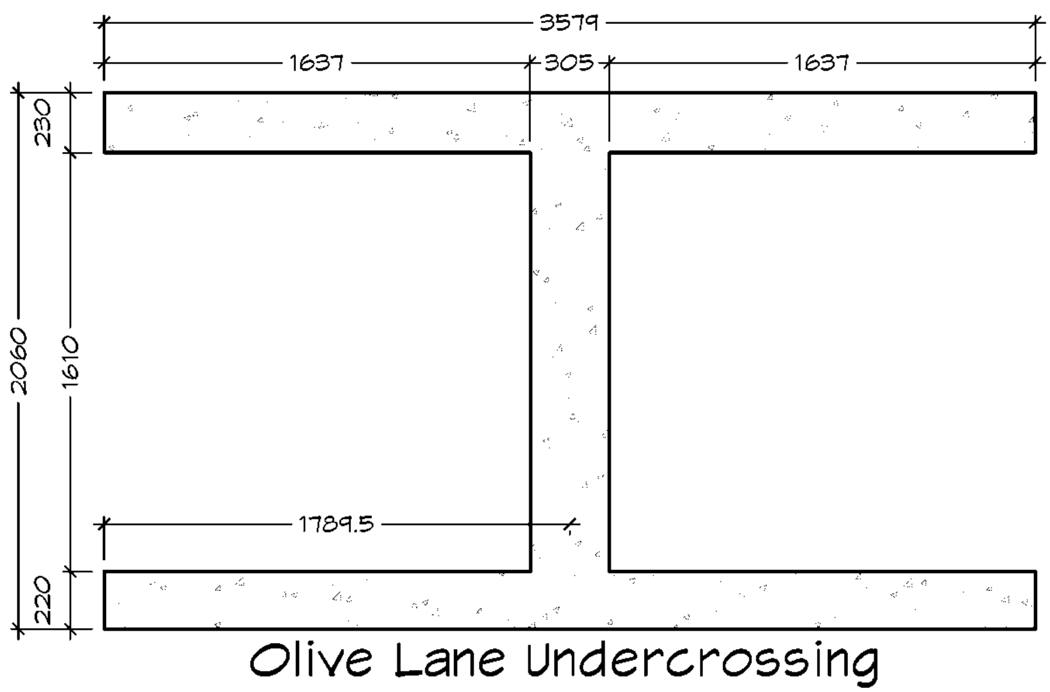


Figure 33. Modeled cross-section for Olive Lane Undercrossing bridge (largest box section; dimensions in mm.)

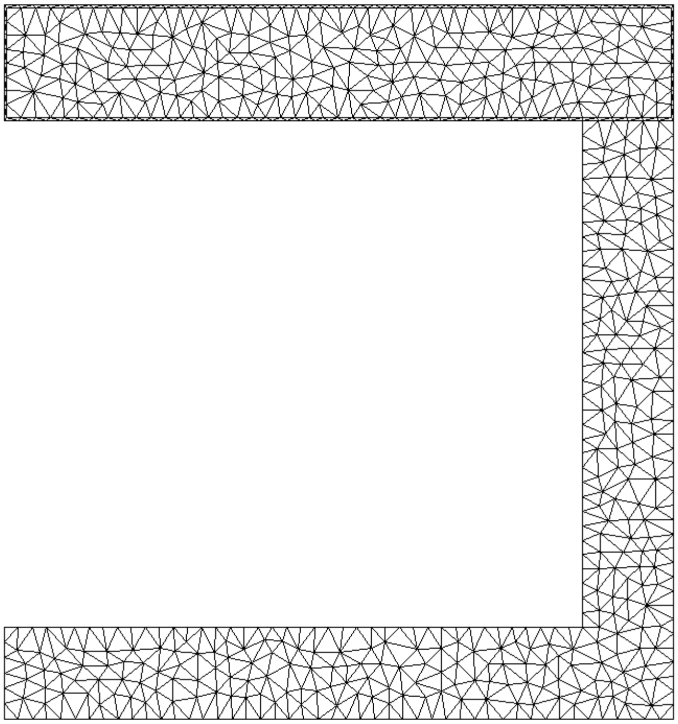


Figure 34. Mesh of symmetric half-section modeled.

Environment

Two different weather environments were examined. For the cooler environment we assumed average May weather in Tahoe City, and for the warmer environment we assumed average July conditions in San Bernardino.

The modeled air temperatures for the cooler environment cycled daily from a low of 35°F (2°C) to a high of 60°F (16°C), with an average of 47°F (8°C). The air temperatures modeled for the warmer environment cycled from a low of 63°F (17°C) to a high of 96°F (36°C), with an average of 80°F (27°C).

The applied solar radiations varied from none at night, to peak afternoon values of 750 and 1300 Watt-hour/m² for the cooler and warmer environments, respectively. Step functions were applied to model the diurnal weather records, with peak values occurring between noon and 1:00 p.m. to match historical weather records.

Modeled wind speeds varied in a step function from 7 mph (3 m/s) at night to 11 mph (5 m/s) during the day. These winds applied cooling to the top surface of the new deck when the deck was not covered by curing media.

Concrete Properties

Two different concretes were modeled: one that represented the current Caltrans mix and had 675 pounds per cubic yard (pcy) of cementitious material (similar to the Markham Ravine Bridge), and a mix having a reduced cementitious content of 550 pcy. Figure 35 shows the modeled heat curve (the heat a unit amount of cement would generate at a constant temperature of 68°F (20°C), per kg of cementitious material. This curve affects the rate and total amount of heat generated, and is affected by the temperature of the mix (i.e., warmer temperatures accelerate the heat generated, and cooler temperatures decelerate it).

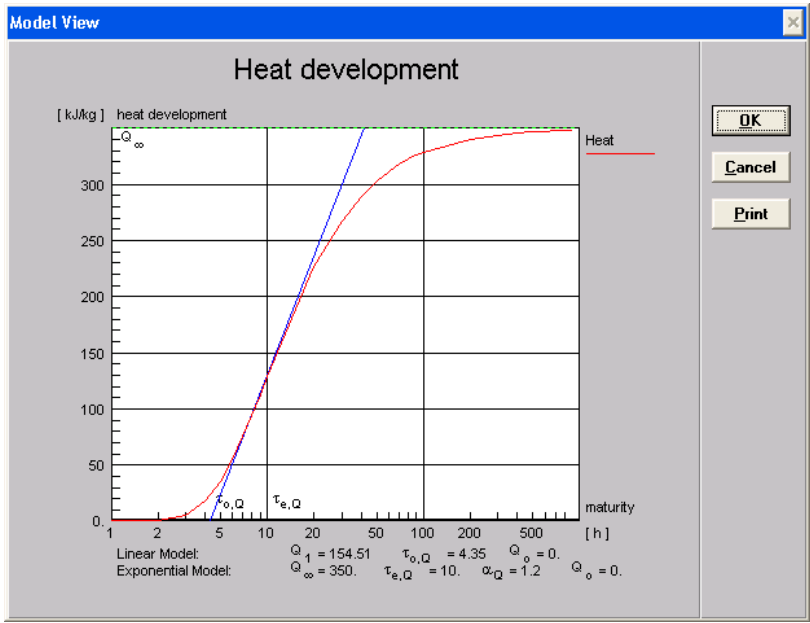


Figure 35. Modeled concrete heat curve.

For all but one set of analyses, the temperature of the concrete at placement matched the average daily air temperature (47°F/8°C for the cooler environment, and 80°F/27°C for the warmer one). For one set of analyses that was the exception, the delivered concrete temperature was 10°F (5°C) cooler than the average daily air temperature in the warmer environment.

Conventional thermal properties were assumed for the concrete. Figure 36 shows the specific properties assumed for the concrete deck in all models.

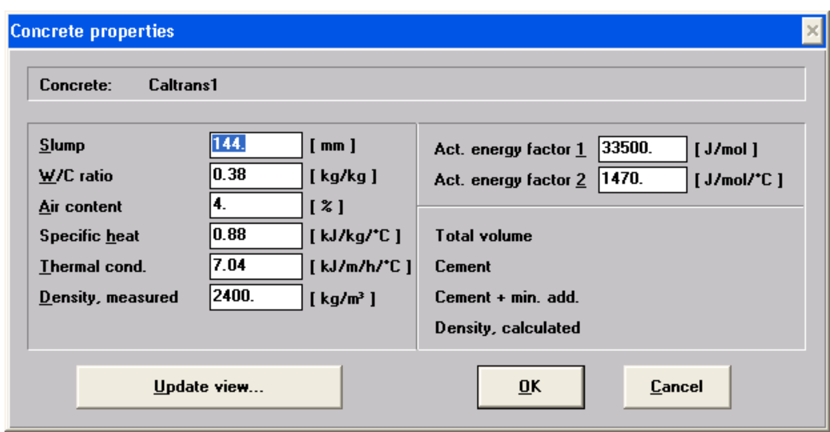


Figure 36. Concrete thermal properties for the deck concrete.

When modeling the time-dependent concrete stiffness, the elastic moduli determined from testing was adjusted by dividing the measured instantaneous elastic modulus by an adjustment factor. This adjustment factor was assumed to vary from 10 at placement, 7 at an age of 6 days, 4 at an age of 24 hours, 3 at an age of 48 hours (2 days), and 2 at an age of 96 hours (4 days). Figure 37 shows the resulting effective concrete modulus of elasticity as a function of time. The elastic modulus affects calculated stresses but not temperatures.

The coefficient of thermal expansion was assumed to be a constant 5.5×10^{-6}/°F (9.9×10^{-6}/°C), and Poisson's ratio was held at 0.2.

The tensile strength used in the models was based upon test data for this project, which yielded the strength curve shown in Figure 38. This curve only affected when cracking was predicted to occur.

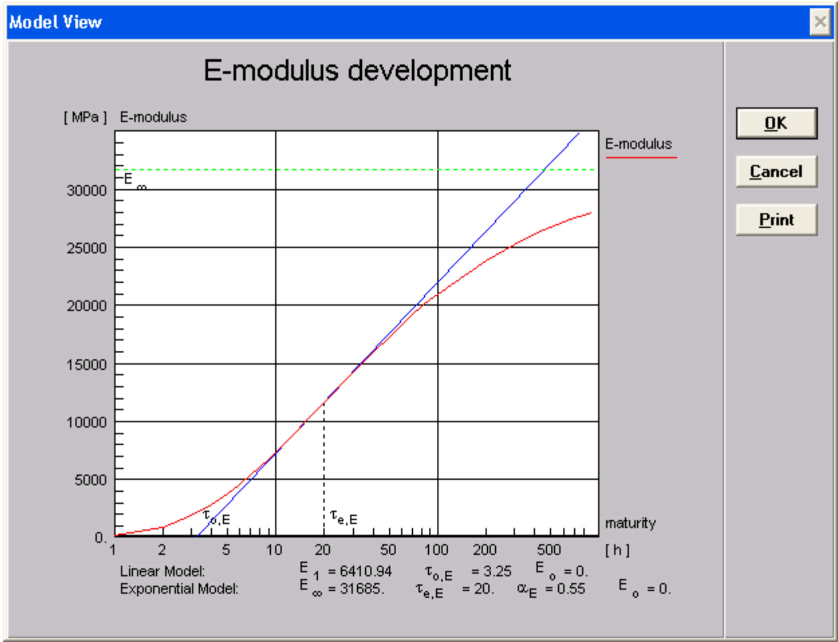


Figure 37. Effective concrete modulus of elasticity (adjusted/softened for estimated creep).

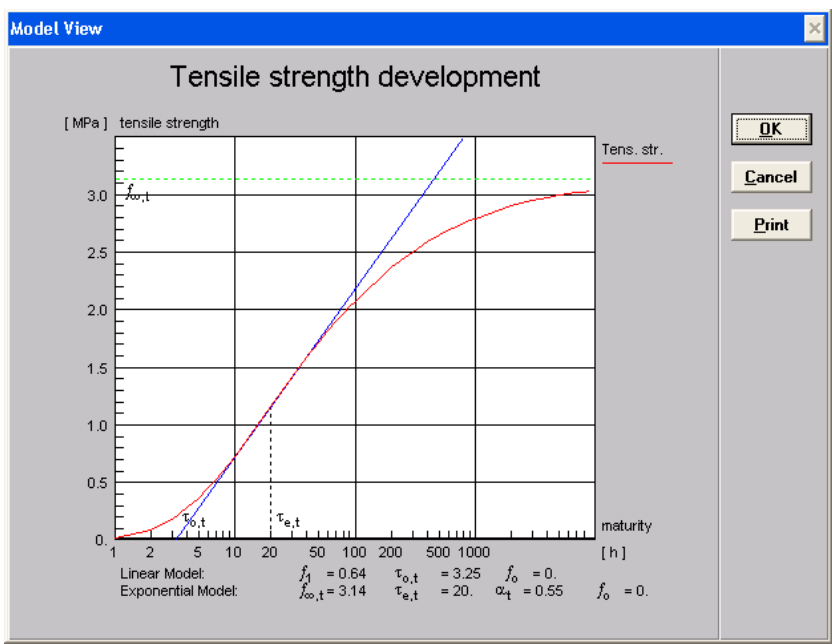


Figure 38. Modeled concrete tensile strength.

Time of Concrete Placement

Four casting times were considered for each analysis set: 6 a.m., 12 p.m., 6 p.m., and 12 a.m. Regardless of the placement time, the temperatures of the delivered concrete and the underlying structure were not changed (47°F/8°C for the cooler environment, and 80°F/27°C for the warmer one).

Other Material Properties

The concrete of the webs and soffits were assumed to be mature and of full stiffness. Figure 39 summarizes the modeled properties of those concrete elements.

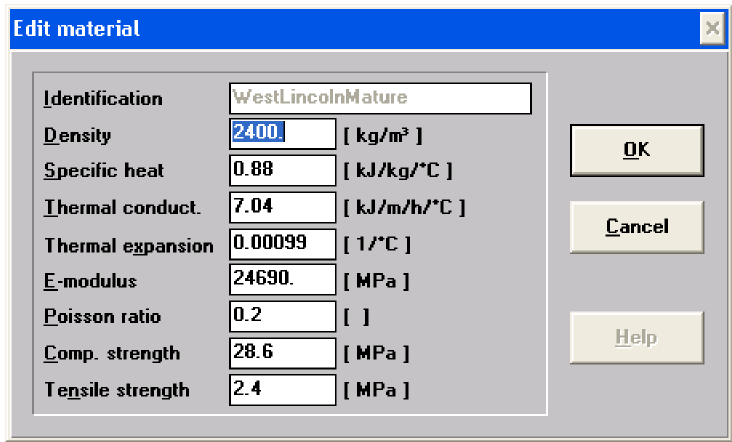


Figure 39. Assumed properties of the concrete webs and soffits

Initial analytical models revealed that heat loss of the deck into the air cavity below it was so small that it affected calculated temperatures negligibly (usually by 1 percent or less). Because modeling the air increased the number of finite elements several fold and thereby increased analysis runtime even more, yet the effects were negligible, the air void below the deck was typically not included in the models.

Calculated Temperatures

Error! Reference source not found. shows one of the heat curves generated for a bridge in the warmer nvironment. Unlike bridge decks cast on steel or concrete girders, the decks cast as part of a box bridge lost heat much more slowly. Heat transfer and loss occurred primarily through the top surface only, with some localized cooling of the deck occurring immediately above the web below it. Little temperature drop typically occurs in the first 48 hours, primary because the heat from continuing hydration essentially offsets cooling into the environment. Because the decks of box bridges tend to lose their heat more slowly, cooling of the deck occurs when the concrete is stiffer (has developed a larger modulus of elasticity) and has less creep potential, resulting in a tendency to develop larger tensile cracking and thereby greater cracking risk. Figure 41 shows a representative temperature profile in the section after 24 hours.

Table 7 summarizes the different combinations of geometry, materials, and conditions examined, along with maximum temperatures and temperature differentials plotted both within 24 hours of casting the deck and for the first four days.

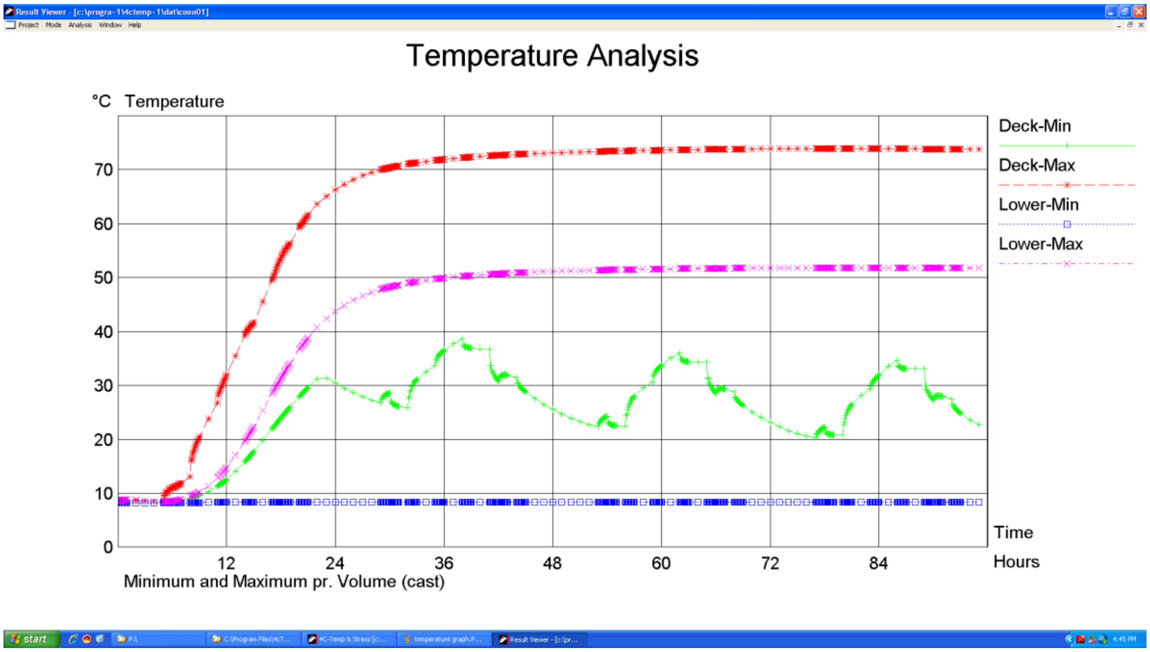


Figure 40. Calculated temperature cycling in the concrete deck and underlying web and soffit (Lower) for midnight pour of delivered concrete temperature of 47°F (8.3°C), with ambient air temperatures cycling between 35 to 60°F (2 to 16°C), no insulation (the maximum deck temperatures plotted in red, the minimum deck temperature plotted in green).

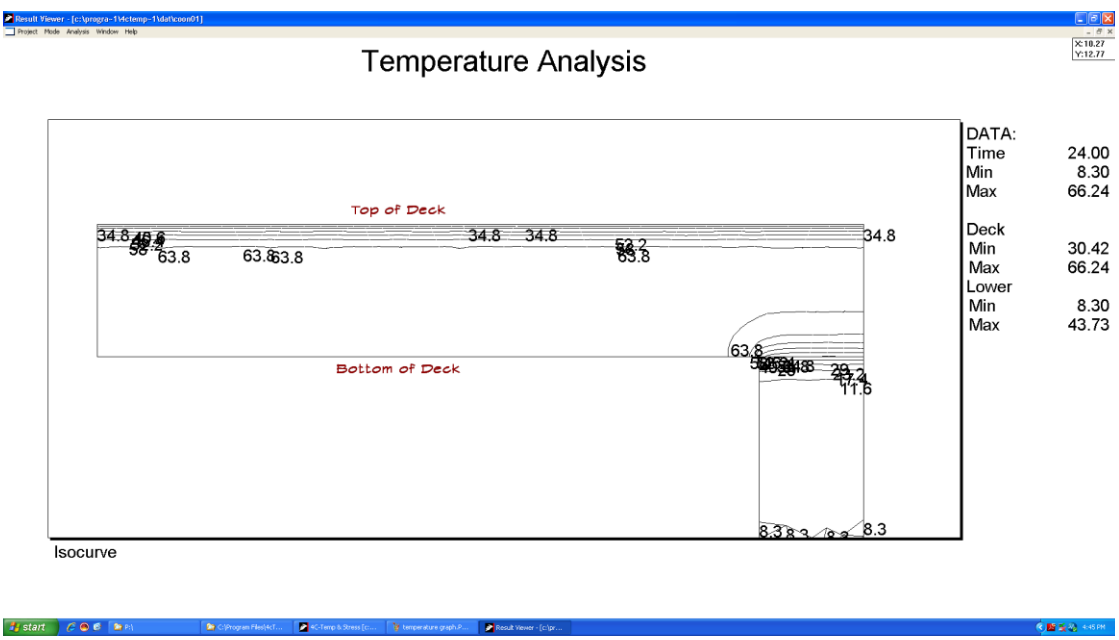


Figure 41. Calculated stress plots for midnight pour of delivered concrete temperature of 47°F (8.3°C), with ambient air temperatures cycling between 35 to 60°F (2 to 16°C).

Table 7 — Summary of Early Thermal Analysis of Box Bridges

Model	Section	Concrete	Temperature: Weather	Temperature: Lower Concrete	Temperature: Delivered Concrete	Temperature: Diurnal Low	Temperature: Diurnal High	Delivery Time	Covering	Peak, °C	Time of Peak (hrs)	First 24 hours: Max. Diff. °C	First 24 hours: Time of Max. Dif., hrs	First 4 days: Max. Diff. °C	First 4 days: Time of Max. Dif., hrs
Coon01nr	Coon Creek	675 pcy cm	Cooler, no radiation	47°F (8.3°C)	47°F (8.3°C)	35°F (1.7°C), no solar radiation	60°F (15.6°C), no solar radiation	midnight	none	73	72	34	24	56	78
Coon02nr								6 a.m.		73	72	38	24	56	74
Coon03nr								noon		74	72	40	24	56	68
Coon04nr								6 p.m.		73	78	38	24	54	78
Coon01			Cooler			35°F (1.7°C)	60°F (15.6°C)	midnight		74	72	37	24	54	77
Coon02								6 a.m.		74	72	41	24	53	71
Coon03								noon		73	66	34	24	52	65
Coon04								6 p.m.		73	66	31	24	51	60
Coon05								midnight	plastic, 6 hours after	74	72	34	24	48	77
Coon06								6 a.m.		74	78	36	24	47	71
Coon07								noon		73	66	34	24	46	65
Coon08								6 p.m.		73	60	32	24	46	60
Coon01w			Warmer	80°F (26.7°C)	80°F (26.7°C)	63°F (17.2°C)	96°F (35.6°C)	midnight	none	92	54	38	24	54	77
Coon02w								6 a.m.		93	54	45	24	55	71
Coon03w								noon		93	72	45	24	54	66
Coon04w								6 p.m.		93	66	37	15	53	60
Coon05w								midnight	plastic, 6 hours after	92	36	38	24	55	77
Coon06w								6 a.m.		93	54	45	46	55	71
Coon07w								noon		93	48	44	20	54	65
Coon08w								6 p.m.		93	54	37	15	53	59
Coon01wc					70°F (21.1°C)			midnight	none	87	60	34	24	50	77
Coon02wc								6 a.m.		87	60	41	24	50	71
Coon03wc								noon		87	60	40	18	49	66
Coon04wc								6 p.m.		87	54	30	24	48	60
Coon05wc								midnight	plastic, 6 hours after	87	60	34	24	50	78
Coon06wc								6 a.m.		87	54	41	24	50	72
Coon07wc								noon		87	60	39	18	49	66
Coon08wc								6 p.m.		87					
Oliv01nr	Olive Lane	675 pcy cm	Cooler, no radiation	47°F (8.3°C)	47°F (8.3°C)	35°F (1.7°C), no solar radiation	60°F (15.6°C), no solar radiation	midnight	none	74	78	36	24	57	78
Oliv02nr								6 a.m.		74	78	38	24	57	74
Oliv03nr								noon		74	72	41	24	57	66
Oliv04nr								6 p.m.		74	78	38	24	56	62
Oliv01			Cooler			35°F (1.7°C)	60°F (15.6°C)	midnight		74	78	37	24	54	77
Oliv02								6 a.m.		74	72	44	12, 24	53	74
Oliv03								noon		74	76	35	24	53	64
Oliv04								6 p.m.		74	66	31	24	52	62
Oliv1cm		550 pcy cm						midnight		62	84	26	24	44	77
Oliv2cm								6 a.m.		62	90	31	24	44	71
Oliv3cm								noon		62	84	27	24	43	66
Oliv4cm								6 p.m.		24	76	24	24	42	59
Coon01a	same as Coon01 except interior air is added									73	72	37	24	54	77

As the data in the summary Table 7 demonstrates, reducing the cementitious material content beneficially affected both peak temperature and temperature differentials more than any other factor. For the Olive Lane bridge, simply reducing the cementitious content from 675 pcy to 550 pcy reduced the peak temperature by 22°F (11°C), reduced temperature differentials in the first 24 hours by 13 to 23°F (6 to 11°C), and reduced temperature differentials in the first four days by about 18°F (9°C). These differences are substantial and demonstrate the strong thermal benefit of simply using less cementitious material in the concrete mixes. Using less cementitious material in decks cast over steel or concrete girders would also be beneficial to those types of bridges, although the benefit would be less than in box girder bridges because of more rapid cooling of the decks.

The concrete deck placed in the warmer environment developed typical temperatures gains and differentials that were approximately ten percent larger than the deck placed in the cooler environment. This is mostly attributable to the concrete being placed in the warmer environment arriving at the site at a warmer temperature, increased solar radiation during warmer seasons, and warmer air temperatures, all accelerating the hydration rate. While a ten percent difference is not large, it is significant and it indicates that a deck cast during summer will have a larger risk of developing cracking than a deck cast in cooler weather. Casting cooler concrete during warm weather also reduced peak deck temperatures and temperature differentials in the deck. Shading the bridge deck, or covering the deck with a white or light-colored material to reflect solar radiation, and other methods to reduce heat from the environment, will reduce the risk of cracking in most bridge decks, especially box bridges.

The time in the day when the concrete was placed had a significant effect on concrete temperatures. Concrete cast in the morning (which occurred on the two bridges monitored for this study) typically developed the largest temperatures and maximum temperature differentials, primarily because peak solar radiation (occurring around noon or very early afternoon) was warming the concrete several hours after placement, just as hydration was beginning in earnest. Air temperatures were also warmer during that time, further heating the concrete mix and accelerating its hydration. Conversely, when the concrete was placed in the late afternoon or early evening, air temperatures are cooling and solar radiation is decreasing or gone when the concrete hydration becomes significant, and for most of the first 24 hours or so the environment is cooling the concrete instead of heating it.

The size of the box girders did not have a large effect on temperatures. This was because the webs transferred relatively little heat out of the deck, reducing the effect that the geometry of the underlying structure had. The calculated temperature profiles suggest that the deck thickness is the largest geometry factor affecting temperatures, with thinner decks developing somewhat smaller temperatures and stresses.

Calculated Stresses

A study of early stresses was performed for several of the analyzed sections. Figure 42 provides a representative plot of one of the models, showing the maximum tensile stress increasing rapidly in the first 36 hours and continuously increasing throughout the 96-hour period examined. Figure 43 shows the calculated stress contours at 24 hours. Calculated tensile stresses were always greatest in the bottom of the deck soffit where the soffit contacted the restraining web. The stresses graphed in Figure 42 are peak corner stresses and substantially over-estimate the more relevant stresses away from the corners (as shown in Figure 43).

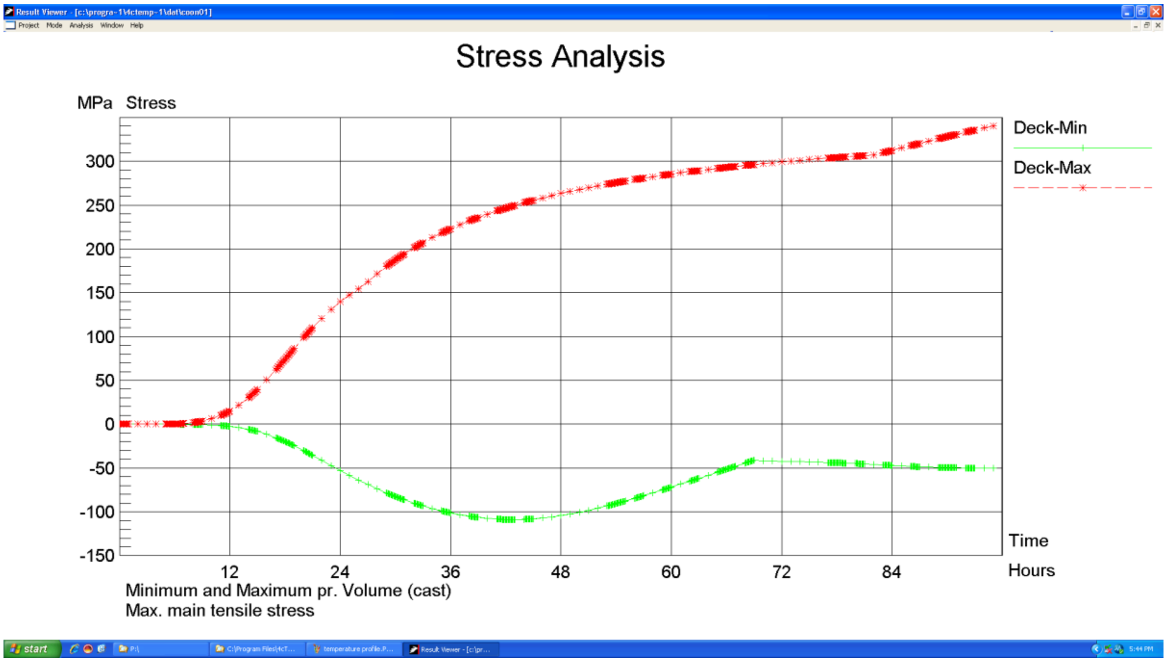


Figure 42. Sample plot of calculated early thermal stresses (MPa) plotted against time.

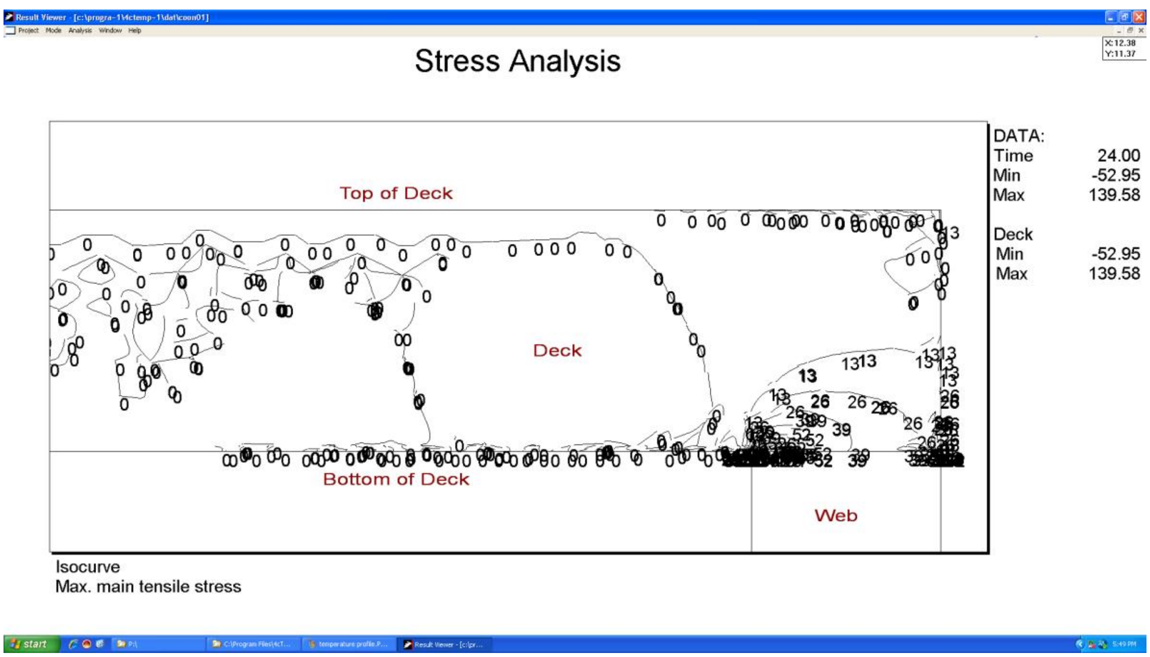


Figure 43. Sample plot of thermal stresses 24 hours after casting.

In brief, the calculated stresses were many times larger than the expected tensile strengths, suggesting that thermal cracking could readily initiate within the first 24 hours. Because the model was limited to elastic behavior only, it did not capture the effects of initial cracking that would relieve localized stresses and soften the concrete, reducing stresses throughout the deck. The high stresses, combined with actual deck performance being significantly better than predicted by analyses, also indicate that much higher creep than expected probably occurs at very early ages.

Of interest and importance is that the calculated stresses continued to increase during the first four days, and that very large temperature differences between the deck and lower portions of the box bridge remained throughout the period. Eventually, as the deck cools, very large thermal stresses are possible, especially because the deck will already have most of its stiffness and will have lost most of its early creep potential, so that the free strains associated with the cooling will correspond to large stresses (generally proportional to the creep-adjusted modulus of elasticity). These temperature differences and resulting thermal stresses are expected to be significantly less in decks cast over steel and concrete girders, since lower peak temperatures typically develop, and cooling occurs much sooner (while the concrete is softer and has more creep potential).

Lattice Modeling of Temperature Changes in Freshly Cast Concrete Bridge Decks

John Bolander, professor at the University of California Davis, performed lattice modeling to simulate deck temperatures and stresses. The primary objective of the Lattice modeling work has been the simulation of temperature changes within freshly cast concrete bridge decks. The simulation approach enables the study of such mitigation strategies as altering the concrete mixture design, lowering of concrete temperature prior to placement, and adjusting the time of deck casting[PK1].

The lattice modeling approach is based on several past works (Bolander, Saito, 1998; Bolander, Berton, 2004; Bolander, Choi, Duddukuri, 2008). The routines used for modeling moisture diffusion and drying from an exposed surface (Bolander, Berton, 2004) are adapted here for thermal analyses. Up to this stage of the analysis efforts, the lattice model has no apparent advantages or disadvantages in comparison with models constructed from low-order finite elements. The lattice model becomes advantageous when concrete cracking is considered.

The following items have been completed for this study:

- Implementation of a heat of hydration model within the lattice modeling of structural concrete. The model has been validated through comparisons with published results and experiments. The heat of hydration model is described in Appendix C, along with validation exercises.

- Extension of the lattice model to account for heat exchange with the environment via convection, solar radiation, and thermal radiation. These extensions, and the parameters used for the thermal analyses of the bridge decks, are described in Appendix C.

- Analyses of temperature variation in the instrumented regions of the Markham Ravine Bridge within the I-65 bypass of Lincoln, CA, and the Olive Lane Undercrossing project in Santee, CA. Good correlation was obtained with the field measurements of temperature for several days after casting of the deck concrete.

- Parametric analyses of temperature variation within the Markham Ravine Bridge deck. The following parameters were studied: amount of solar radiation on the day of casting, cementitious materials content, casting time, and temperature of the fresh concrete at the time of casting.

Analyses of Temperature Variation within the Markham Ravine Bridge

Model Configuration and Input Quantities
The lattice model described in Appendix C is used to simulate temperature development within the instrumented regions of the Markham Ravine Bridge deck. Discretizations of a longitudinal segment of the bridge structure are shown in Figure 44 and Figure 45. The freshly placed deck concrete produces heat due to hydration, while the supporting concrete acts as a heat sink. To enable heat exchange with the environment, the surrounding air and the space within the internal cells of the deck have also been discretized. Nodes were placed at the locations of the temperature sensors, both at the mid-span and web locations (Figure 45), so that comparisons can be made between the simulated and measured temperature.

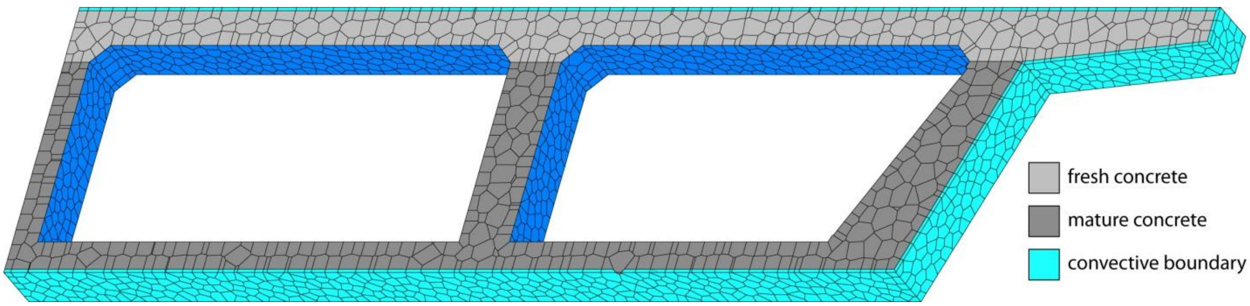


Figure 44. Lattice model of longitudinal segment of the Markham Ravine Bridge. Symmetry about the bridge centerline has been exploited. Discretization of the enclosed air space is shown in Figure 45.

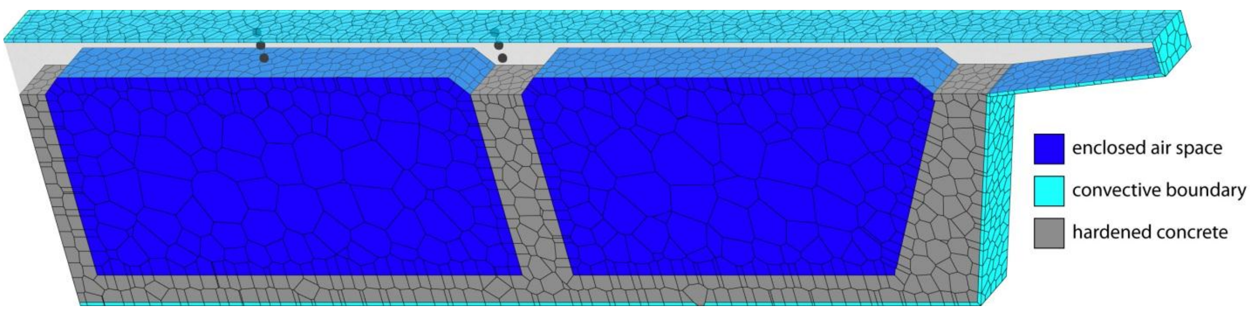


Figure 45. Another view of the bridge segment discretization and indication of the thermocouple locations.

For the assumed geometry and boundary conditions, there is no heat transfer in the longitudinal direction of the bridge and therefore a two-dimensional model would have been sufficient within the limits of this study. However, the three-dimensional analysis framework is essential for the ultimate goal of determining the cracking potential of the concrete deck.

For the bridge deck analyses, the treatment of boundary conditions within the lattice model was extended to include the effects of variations in ambient temperature, heating due to solar radiation, and cooling due to thermal radiation. Formulations of these boundary conditions are given in Appendix C.

The lattice modeling of temperature variation within the bridge deck system assumes the concrete to be a homogeneous material. Model inputs (e.g., heat capacity, thermal conductivity, and density of the constituents) must ultimately lead to concrete properties in an average sense. To the extent possible, these and other inputs to the lattice model were based on measured or commonly assumed properties. In particular:

- Calorimetry measurements of the concrete mixture were not taken, so the heat of hydration properties of the cement are based on those of cements of similar chemical composition. Whereas the pozzolanic reaction of fly ash is not significant within the first 72 h of hydration [Paine, Zheng, Dhir, 2005], fly ash presence promotes the reaction of portland cement. As a rough approximation, the portland cement/fly ash blend is modeled as a single phase with 50% of the fly ash mass counted toward heat production. The reactions of the Portland cement and fly ash can be modeled separately [de Schutter, Taerwe, 1995; Paine, Zheng, Dhir, 2005], using the technique outlined in Appendix C for each phase.

- The heat capacity of the cement paste is modeled using the approach given by Bentz [2007]. As the hydration process consumes water, heat capacity of the cement paste is a function of degree of reaction of the cement. The heat capacity of the concrete is then determined from the heat capacities of the cement paste and aggregates, according to the mass fractions of each using an ordinary rule of mixtures [Bentz, 2007].

- Thermal conductivity of the concrete is estimated by taking the average of the Hashin-Shtrikman bounds for a two-phase composite formed of paste and aggregates [Bentz, 2007]. The presence of reinforcing bars within the bridge deck has not been considered in these simulations.

- Ambient temperature and wind speeds were obtained directly from on-site measurements. Wind speed measurements were taken for the first four days after the Markham Ravine Bridge deck pour.

- Incident solar radiation, dew point temperature, and cloud cover were obtained from the National Solar Radiation Data Base for the same time period (April 14 – 18), albeit for an earlier year [National Solar Radiation Data Base]. Following Bentz [2000], solar absorptivity of the concrete is taken as $\gamma_{abs} = 0.65$. Additional details are given in Appendix C.

- A combination of burlap and plastic membrane (Transgard 4000) was used to cover the bridge deck at about t = 6 h after concrete placement. The presence of this cover certainly affects heat exchange by both convection and radiation. The product sheet for Transgard 4000 indicates a light reflectance of 0.85. This is roughly twice the reflectance values of ordinary portland cement concrete, which range from about 0.34 to 0.48 [Marceau, Vangeem, 2008]. For this reason, q_{sun} was reduced by a factor of 0.5 for t > 6 h. For lack of information, the same reduction factor of 0.5 was applied to both q_{conv} and q_{sky} for t > 6 h (Appendix C).

Model Results

The simulated temperature histories are compared with the field measurements in Figure 46 through Figure 48. The recorded ambient temperature history is also plotted in the first two figures. Several comments can be made.

- The influence of environmental factors is evident from the oscillatory behavior of the temperature history recorded by each thermocouple sensor. After the first day, locations closer to the surface exhibit the larger temperature swings, whereas the deeper locations are less affected by environmental changes. For example, the amplitude of the daily temperature swings at TC1 are roughly 5 to 8°F (3-4°C) larger than those at TC3. This behavior, which appears in both the model and measured results, meets expectations.

- Peak temperatures occur at about 10 hours after concrete placement. These temperatures are significantly higher than the ambient temperature. Over time, the differences between the ambient and measured deck temperatures diminish.
- The temperature difference between the fresh concrete and the supporting girder is greatest near the time of peak temperature in the fresh concrete (Figure 46). Much of this difference appears early in the temperature history, even before the concrete has set. A potential contribution to cracking comes from cooling of the stiffening concrete in the presence of girder restraint.
- The model results agree fairly well with the measured values, especially considering the various inputs to the model. The model results are sensitive to most of these inputs. Better agreement could have been obtained through a fitting process.
- The gradual increase in daily ambient temperatures makes the deck concrete readings more difficult to interpret. For example, the successive peak temperatures would likely not be as pronounced if the daily temperature did not rise over the four-day interval considered. The ambient temperatures recorded at the Olive Lane Undercrossing in San Diego County exhibit less variation.

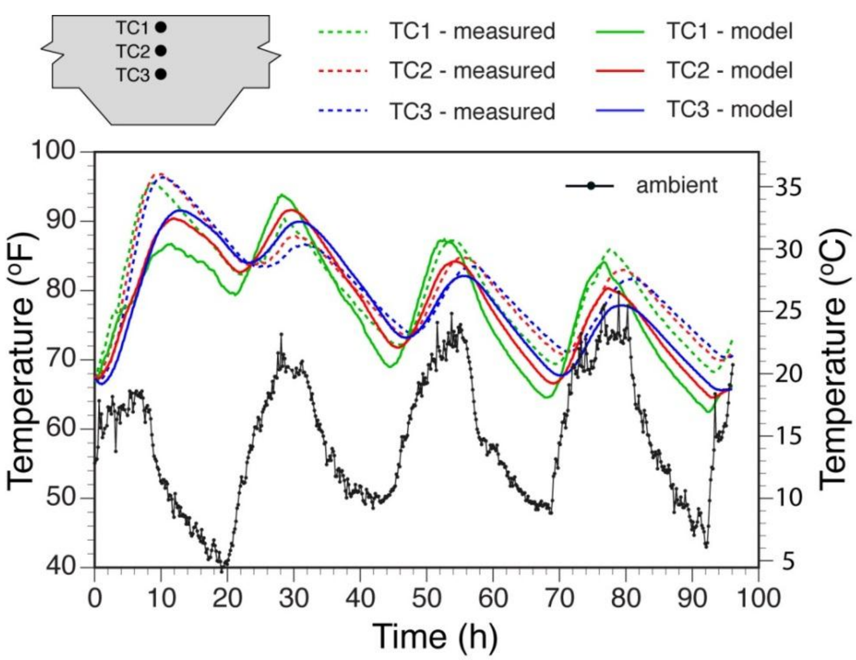


Figure 46. Temperature variation at thermocouple locations within web region of Markham Ravine Bridge (TC1, TC2, and TC3 are positioned at 1, 3, and 7 inches below the deck surface, respectively).

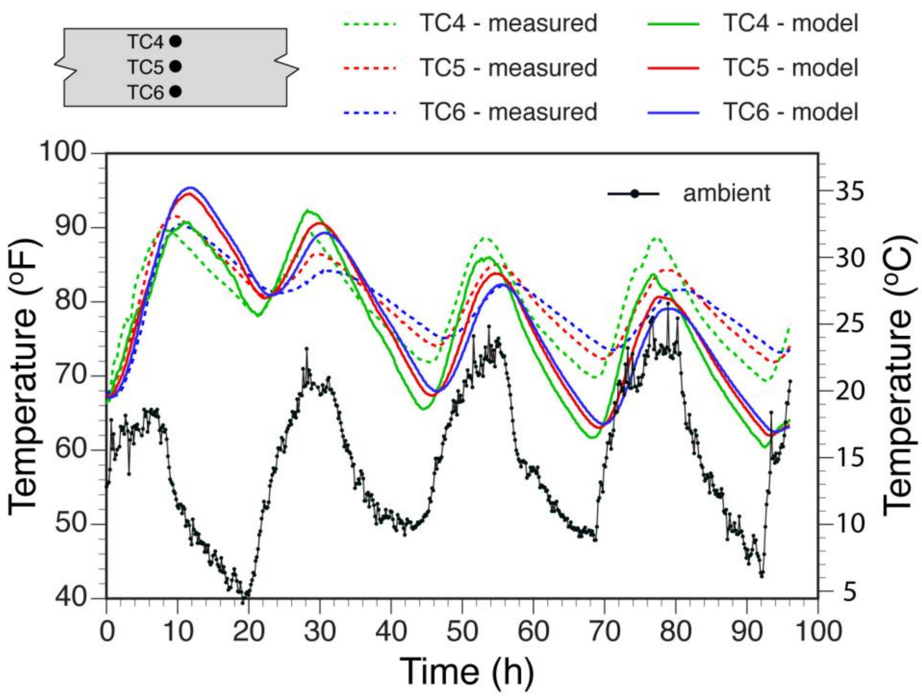


Figure 47. Temperature variation at thermocouple locations within midspan region of Markham Ravine Bridge (TC4, TC5, and TC6 are positioned at 1, 3, and 7 inches below the deck surface, respectively).

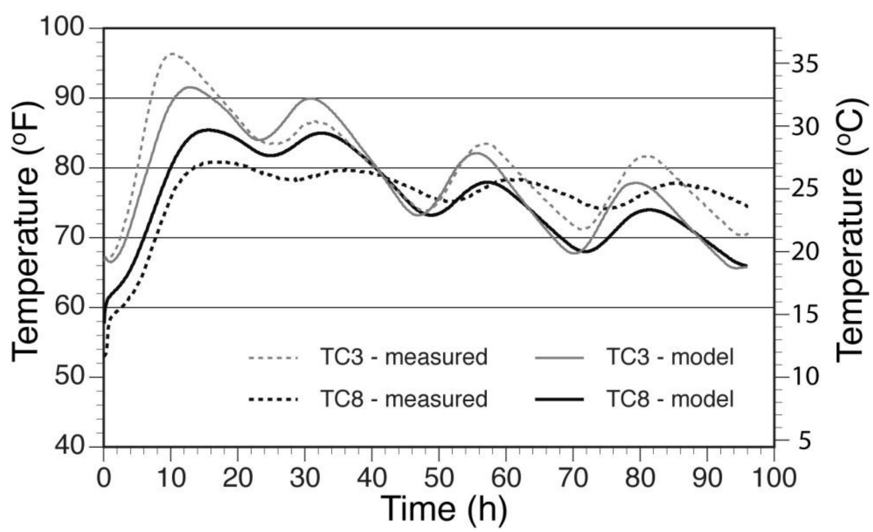


Figure 48. Temperature variation at thermocouple locations within the web and supporting girder (TC3 and TC8 are positioned at 1 inch above and 1 inch below the girder-deck construction joint, respectively)

Parametric Analyses of Bridge Deck Temperature Evolution

The lattice modeling of cement hydration has been validated, at least to some degree, through comparisons with results presented in the literature and the thermocouple readings taken at the Markham Ravine Bridge site (Figure 46, Figure 47, and Figure 48). The model is now used for parametric study of the following influences on peak concrete temperature and temperature difference between the fresh concrete and the supporting concrete structure.

- Solar radiation on the day of casting (base case: sunny on the day of casting)
- Cement content (base case: 590 lb/yd^3)
- Casting time (base case: 10:00 a.m.)
- Fresh concrete temperature at time of casting (base case: 67°F/19°C)
- Concrete mixture design and bridge environment (base case: Markham Ravine bridge project)

The Markham Ravine Bridge is used for these parametric analyses. As for the previous analyses of the Markham Ravine Bridge, the other model inputs have been taken from on-site measurements, the concrete constituents and mixture design, and typical values cited in the literature.

Solar Radiation

Analyses were run using solar radiation data for two different days during the project time period, albeit for an earlier year. The data was obtained from the National Solar Radiation Data Base and is presented in Figure 49. The two plots in the figure might represent, in rough terms, a cloudy and sunny day, respectively.

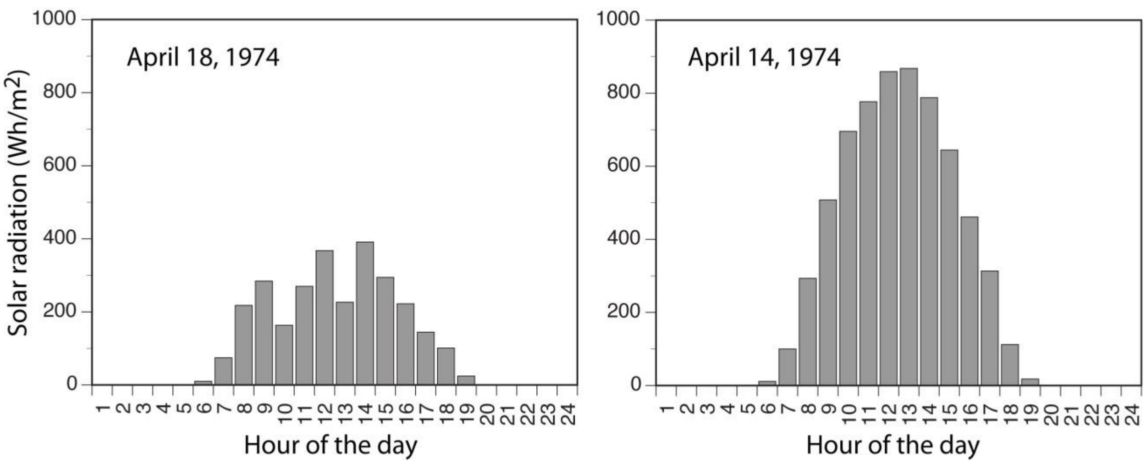


Figure 49. Amount of direct and diffuse solar radiation received on a horizontal surface during the 60 minutes preceding the hour indicated [National Solar Radiation Data Base]

Figure 50 shows the variation of temperature at locations TC3 and TC7 for these two exposure conditions on the day of casting. Increased solar input for the second case results in an increase of 12°F (6°C) in the peak temperature at TC3 and about a 14°F (7°C) increase in the maximum temperature difference between TC3 and TC7. These temperature increases are due not only to solar input, but also to increased rate of cement hydration at higher temperatures. After the first day, the solar radiation conditions for both cases were assumed to be the same (and equal to those used for the preceding analyses of the Markham Ravine Bridge).

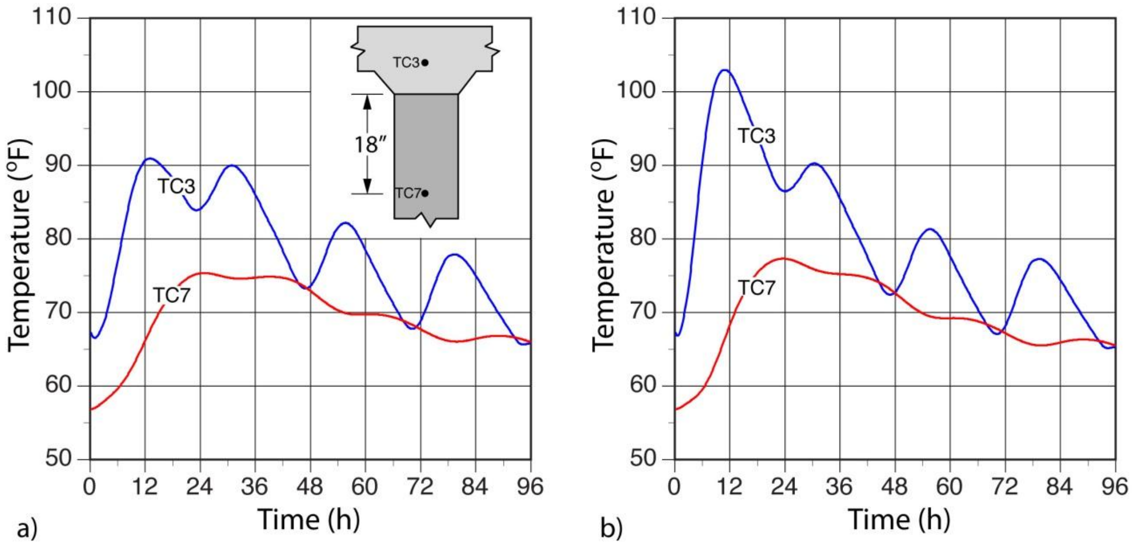


Figure 50. Simulated temperature variations in deck web and supporting girder: a) cloudy first day; and b) sunny first day.

Cement Content

Simulations were run for three different cement contents: 506, 590.5, and 675 lb/yd^3. The first of these amounts corresponds to the actual Portland cement content of the Markham Ravine Bridge mixture; the latter two amounts correspond to 50% and 100% of the actual fly ash addition counting as Portland cement. Figure 51 presents results for peak temperature and maximum temperature difference between TC3 and TC7 for the two different exposure conditions to solar radiation. The temperature increase with increase in cement content (i.e., the slope of each trend line) is greater for the condition of higher solar radiation input. As expected, the most undesirable case is the combination of high cement content and high solar radiation input, resulting in a temperature difference of about 44°F (22°C).

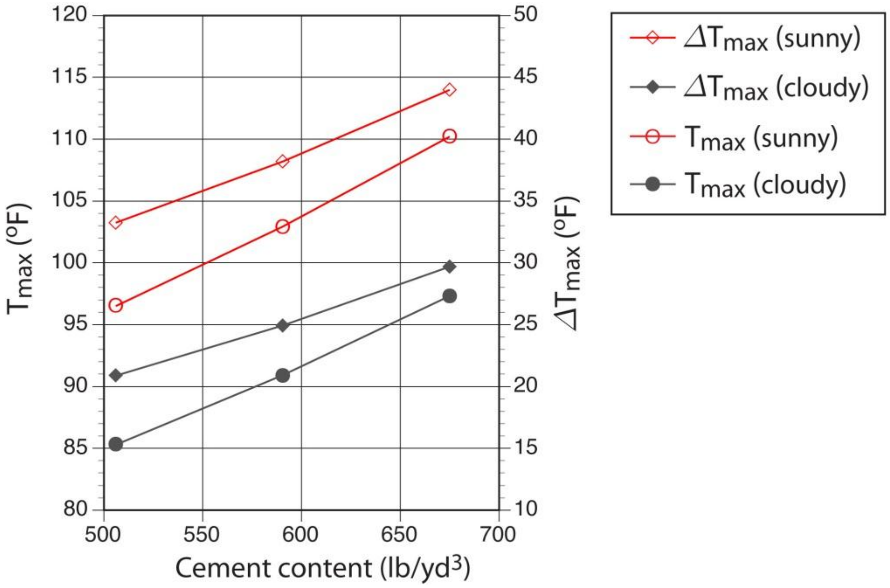


Figure 51. Effect of cement content and solar radiation on maximum temperature in deck web and temperature difference between web and supporting girder.

Casting Time

Temperature development in the bridge deck is simulated for a range of casting times, as shown on the horizontal axis of Figure 52. Whereas some of these casting times might not be practical, the range of times provides a picture of the thermal behavior of the deck/structure system. The results in Figure 52 tend to show that afternoon casting times are preferable in terms of reducing the temperature difference between the fresh deck concrete and the supporting structure. With a 4:00 p.m. casting time, for example, the fresh concrete receives little of the day-one heat input associated with solar radiation (Figure 49). For the Markham Ravine Bridge, ambient temperature dropped sharply after 6:00 p.m., so heat exchanges associated with convection and thermal radiation were also favorable. Care should be taken when extrapolating these results to conditions where evening cooling is not significant.

For casting times of 4:00 p.m. or earlier, the peak temperature difference occurs during the first upward temperature swing (which occurs during the first day and possibly into the early morning hours of the next day). For casting times after 4:00 p.m., the peak difference occurs on the second upward temperature swing, which is associated with solar and convective heating during the second day.

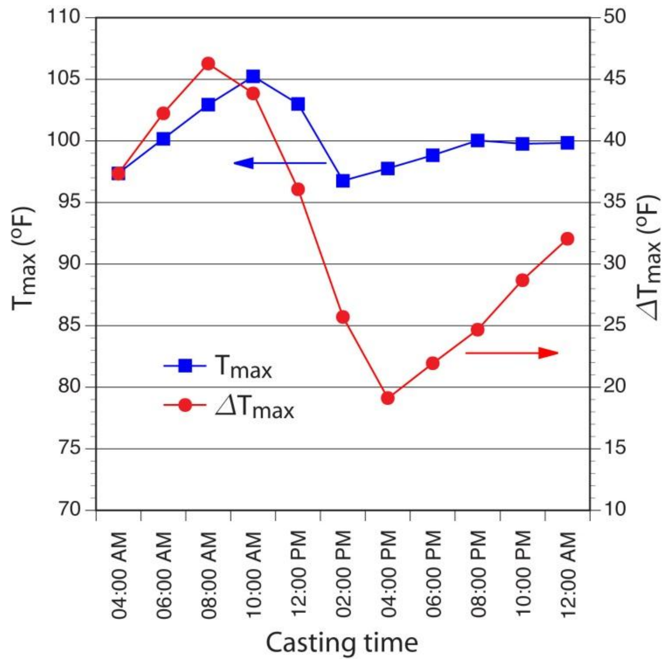


Figure 52. Influence of casting time on peak temperature and maximum temperature difference.

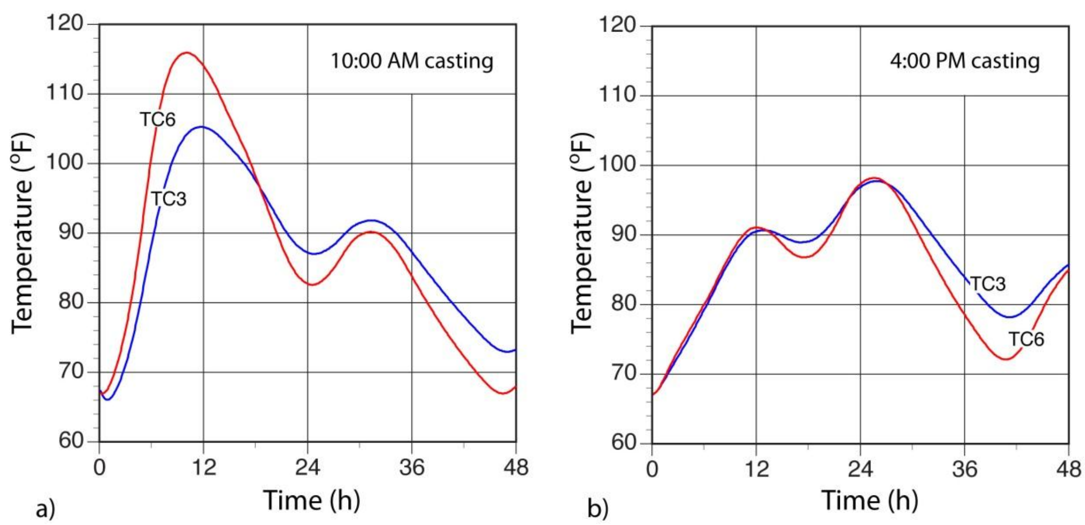


Figure 53. Simulated temperature variation in deck from time of casting: a) 10:00 a.m. casting; and b) 4:00 p.m. *casting (TC3 above web; TC6 in deck mid-span).*

The bulk of the results in this section are for temperatures in the deck above the web. Figure 53 compares such temperatures (at TC3) with those at mid-span between girders (at TC6). For morning casting times, the deck between girders becomes significantly hotter than directly above the girder. Solar heating of the concrete above the web is tempered by heat conduction toward the cooler concrete of the supporting girder. Less heat is transferred to the air in the enclosed cell beneath the mid-span location, so that location exhibits higher peak temperatures. The field measurements for Markham Ravine Bridge did not demonstrate this pattern, possibly due to the presence of cloud cover on the day of casting.

Fresh Concrete Temperature at Time of Casting

Simulations are run for temperature variations of +/-10°F (-12°C) about the base case of 67°F (19°C). Peak temperatures are most affected by morning casting times, as shown in Figure 54. For afternoon and evening casting times, net heat exchange with the environment is smaller (in an algebraic sense) and so higher temperature of the fresh concrete has less influence on peak temperature.

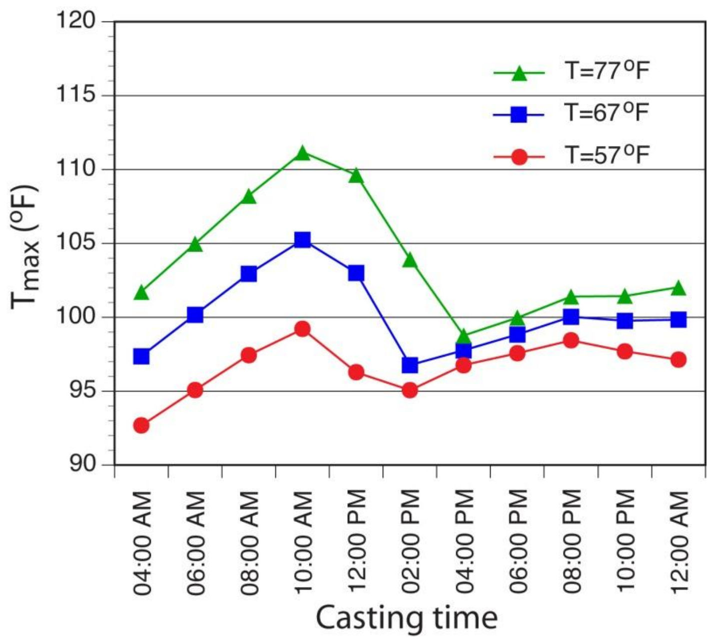


Figure 54. Influence of initial temperature of fresh concrete on deck peak temperature.

When considering peak temperature difference, the effect of fresh concrete temperature is lessened by heat conduction from the fresh concrete into the supporting structure (Figure 55). For later afternoon and evening casting times, the normal trend reverses: higher fresh concrete temperatures result in slightly lower maximum temperature differences (due to warming of the supporting concrete by heat conduction from the fresh concrete).

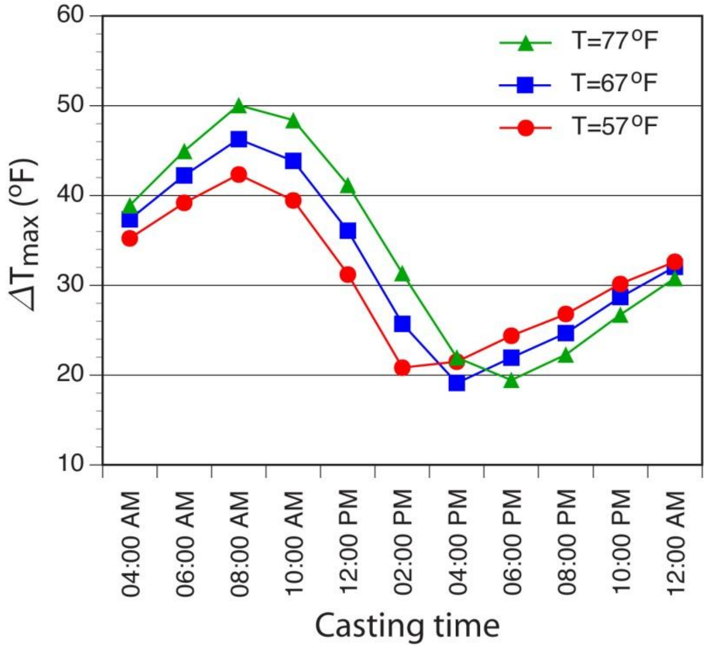


Figure 55. Influence of initial temperature of fresh concrete on the maximum temperature difference between deck and supporting structure

Concrete Mixture Design and Bridge Environment

The concrete mixture design and environmental conditions of the Olive Lane Undercrossing project were simulated. In lieu of constructing a new mesh, the mesh developed for the Markham Ravine Bridge (Figure 44 and Figure 45) was used for modeling temperature evolution. Comparing dimensions of the two structures, the Olive Lane Undercrossing deck is only 0.60 in. (15 mm) deeper and its girder is only .20 in. (5 mm) wider.

The mixture design and environment conditions of the Olive Lane Undercrossing project differ in several notable respects, in comparison to those of the Markham Ravine Bridge:

- The Portland cement and fly ash contents are 11.5 and 11.2% greater by weight, respectively. These increases were made mainly through a reduction in the amount of coarse aggregate; and

- The average first-day temperature is roughly 20°F (10°C) warmer.

Simulation results are compared to the field measurements in Figure 56. The two sets of results agree well in a qualitative sense. The TC1 readings are the first to climb but reach the lowest peak temperatures; the TC3 readings are the last to climb but ultimately yield the highest peak temperatures. As for the Markham River Bridge, the simulated temperatures over the web region are lower than the ones recorded in the field. These differences could have been reduced through adjustment of the model settings, but such fitting exercises were not the objective of this work. Rather, to the extent possible, the model settings were based on the field measurements, concrete mixture constituents and their proportions, and typical values provided by the literature.

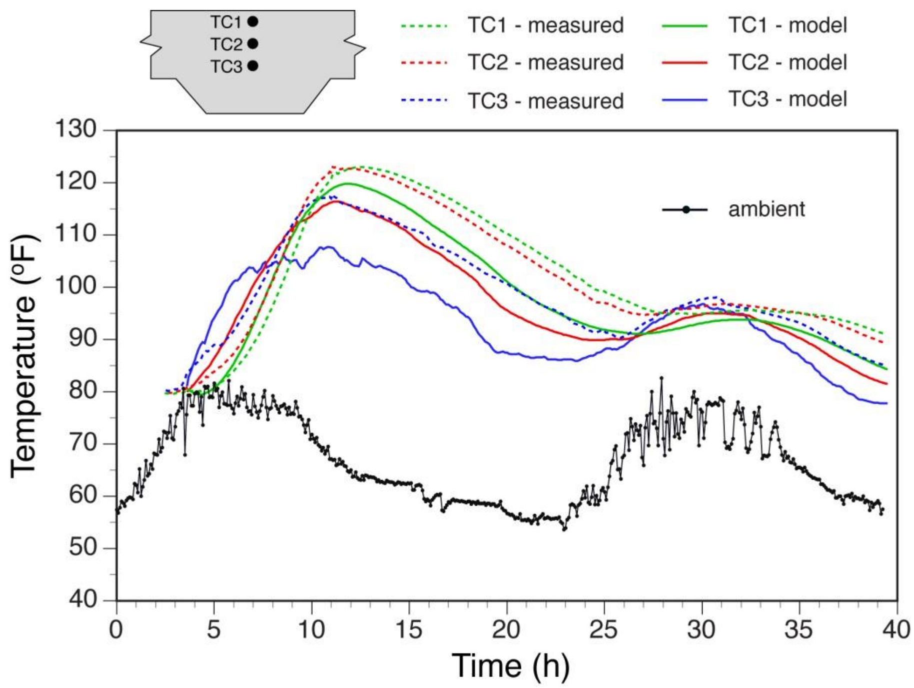


Figure 56. Temperature evolution at thermocouple locations within web region (for the concrete mixture design and environmental conditions of the Olive Lane Undercrossing project).

Summary

The temperature difference between the new deck concrete and the supporting girders reached up to 44°F (24°C) for the parameter combinations considered in this report. The closing of this temperature gap, during cooling of the increasingly mature deck concrete, is a potential contributing factor to deck cracking.

The field measurements and simulation results confirm expectations: temperature variation within the new concrete deck is sensitive to a number of environmental factors. Due to the strong physical bases of the model, the simulations are potentially useful for predicting how environmental factors (such as variations in ambient temperature, solar radiation, and wind speed), concrete mixture design, and placement/curing strategies affect temperature development.

The parametric analyses have indicated that solar heating, cement content, time of concrete casting, and concrete temperature at the time of casting are all primary factors affecting concrete peak temperature and maximum temperature difference between the fresh deck concrete and the supporting concrete structure. The results suggest a number of potential ways for controlling temperature differences that arise due to cement heat of hydration and heat exchange with the environment and supporting structure. At least for the parameter settings considered here, afternoon casting times are preferable to morning casting times. Solar heating, cement content and initial temperature of the fresh concrete should be reduced if possible.

Summary of Parameter and Lattice Numerical Modeling

As previously noted, elastic theory tells us that uniform or linear temperature changes and shrinkage do not cause stresses when the movements (strains) naturally associated with temperature changes and shrinkage are not restrained. For uniform or linear temperature changes, stresses develop only when the free strains are restrained, and the magnitudes of stresses depend upon the amount of restraint. For a concrete box girder, the webs and web provide the primary restraint; for other bridges, the beams provide the primary restraint. Additional stresses develop from nonlinear temperature changes, especially during the first 24 or 48 hours when temperature changes are highly nonlinear; and later on as the concrete dries from its surface while interior regions have greater humidity, resulting in non-uniform drying shrinkage.

Most of the time, the bridge deck and the underlying structural elements will have different temperatures, and different shrinkage relative to when the deck was cast. Consequently, the deck and underlying structure would have different lengths if not connected together. Because they are connected together, forces develop between the deck and underlying structure to maintain the same strains in each at the interface. These forces are not at the centroids of the elements, and consequently curvature develops in the bridge spans. To maintain equal curvature across the interface, moment couples develop at the ends. (Krauss and Rogalla, 1996).

Curvatures (from temperatures and shrinkage differences) result in deflections between the end supports. These curvatures do not affect stresses in a single-span bridge, because the bridge is free to deflect between its ends. In a continuous-span bridge, however, the interior supports prevent the beam from deflecting, and the interior supports create additional restraint forces in the bridge. Because concrete decks are placed after the webs and bottom soffits are cast, early temperature changes and shrinkage of the deck typically create a concave-upwards curvature of the girder. The interior supports then apply an upward force against the girders, to maintain the vertical position of the supports. These upward forces create negative bending in the girders around the supports, which results in tension stresses in the deck. These stresses are additive to the stresses resulting from temperature and shrinkage differences alone. As such, transverse deck cracking is often worst at interior supports of a continuous-span bridge.

In a single-span bridge, stresses from temperatures and shrinkage differences generally do not vary substantially along the length of the span; stresses in a continuous-span bridge are the same with the exception of the effects added by the interior supports. The stresses required to maintain compatibility (no slip and no curvature difference) across the deck and web interface develop at the ends of the girder and are generally well developed within a length that extend from the girder end to a length away from it that is approximately equal to the depth of the girder. Between the development zones at each ends, which is most of the span length, the interior stresses from temperatures and shrinkage are generally constant along the length, provided that temperatures and shrinkage differences also do not vary along the length.

The stress in a bridge deck from temperature and shrinkage difference depends largely upon the restraint provided to the deck by the underlying bridge elements. Deeper box girders are stiffer than shallower ones, and as such, restrain more temperature and shrinkage movements in the deck. Similarly, deeper beams are typically stiffer than shallower beams, and thereby will increase restraint applied to the deck. Greater restraint creates larger tensile stresses in the deck, which can cause deck cracking if large enough.

Based on the parameter study, the maximum tensile stresses in the deck always occurred on the bottom (soffit) surface of the deck. Deck stresses were largest when the relative stiffness between the deck and the web and soffit (that provided the deck restraint) was largest. In other words, deck stresses are largest, and the cracking tendency is greatest, when the deck is thinnest, the overall box section is deepest, and the web spacing is narrowest.

Of all the geometry factors examined, the overall height of the box girder had the largest influence on deck stresses. However, because the span length largely determines the required girder depth, modifying the girder depth to reduce the risk of deck cracking may not be practical.

The deck thickness did not have a large effect on deck stresses when the concrete was very young (when the concrete had a small modulus of elasticity and high creep potential), but it had a moderate effect when long-term shrinkage was considered. Similarly, increasing the web spacing did not substantially affect stresses when the concrete was very young, but had a moderate effect long-term.

Initial analytical models revealed that heat loss of the deck into the air cavity below it was so small that it affected calculated temperatures negligibly. Unlike bridge decks cast on steel or concrete girders, the decks cast as part of a box bridge lost heat much more slowly. Heat transfer and loss occurred primarily through the top surface only, with some localized cooling of the deck occurring immediately above the web below it. Little temperature drop typically occurs in the first 48 hours, primary because the heat from continuing hydration essentially offsets cooling into the environment.

Reducing the cementitious material content beneficially affected both peak temperature and temperature differentials more than any other factor. For the Olive Lane bridge, simply reducing the cementitious content from 675 pcy to 550 pcy reduced the peak temperature by 22°F (11°C), reduced temperature differentials in the first 24 hours by 13 to 23°F (7-11°C), and reduced temperature differentials in the first four days by about 18°F (9°C). These differences are substantial and demonstrate the strong thermal benefit of simply using less cementitious material in the concrete mixes.

Casting a deck cast during summer will have a larger potential risk of developing cracking than a deck cast in cooler weather by approximately 10 percent. Casting cooler concrete during warm weather also reduced peak deck temperatures and temperature differentials in the deck.

The time in the day when the concrete was placed had a significant effect on concrete temperatures. Concrete cast in the morning (which occurred on the two bridges monitored for this study) typically developed the largest temperatures and maximum temperature differentials, primarily because peak solar radiation (occurring around noon or very early afternoon) was adding heat to the concrete several hours after placement, just as hydration was beginning in earnest. Air temperatures were also warmer during that time, further heating the concrete mix and accelerating its hydration. Conversely, when the concrete was placed in the late afternoon or early evening, air temperatures are cooling and solar radiation is decreasing or gone when the concrete hydration becomes significant, and for most of the first 24 hours or so the environment is cooling the concrete instead of heating it. As expected, the most undesirable case is the combination of high cement content and high solar radiation input. Afternoon casting times are preferable in terms of reducing the temperature difference between the fresh deck concrete and the supporting structure. With a 4:00 p.m. casting time, for example, the fresh concrete receives little of the first day's heat input associated with solar radiation. For casting times after 4:00 p.m., the peak difference occurs on the second upward temperature swing, which is associated with solar and convective heating during the second day. Cooling the fresh concrete is more important when concrete is cast in the morning than when cast in the late afternoon. For afternoon and evening casting times, net heat exchange with the environment is relatively smaller and so higher temperature of the fresh concrete has less influence on peak temperature.

The size of the box girders did not have a large effect on temperatures. This was because the webs transferred relatively little heat out of the deck, reducing the effect that the geometry of the underlying structure had. The calculated temperature profiles suggest that the deck thickness is the largest geometry factor affecting temperatures, with thinner decks developing somewhat smaller temperatures and stresses.

In brief, the calculated stresses were many times larger than the expected tensile strengths, suggesting that thermal cracking could readily initiate within the first 24 hours.

Recommendations Based on the Parameter and Lattice Numerical Modeling

- Reducing the cementitious material content beneficially affected both peak temperature and temperature differentials more than any other factor. Reducing the cementitious content from 675 pcy to 550 pcy will reduce the risk of deck cracking.

- Casting decks in cooler weather is beneficial and may reduce the potential for risk of cracking by up to 10 percent. Shading the bridge deck, or covering the deck with a white or light-colored material to reflect solar radiation, and other methods to reduce heat from the environment, will reduce the risk of cracking in most bridge decks, especially box girder bridges.

- Casting decks in the afternoon and early evening will often reduce peak temperatures and stresses. The optimum time to reduce the risk of cracking appears to be around 4 p.m. and the hours afterwards, so that the environment is cooling the concrete when its greatest rate of hydration (and heat generation) is occurring, reducing peak concrete temperatures and temperature differentials within the concrete. Conversely, the worst casting time is often around 10 am (unfortunately, when most contractors cast decks), so that the warmest air temperatures and maximum solar radiation are occurring several hours later when the concrete hydration rate is greatest, increasing peak concrete temperatures and temperature differentials within the concrete.

- Continuous-span bridges have significantly greater risk of cracking over interior supports. To reduce the risk of deck cracking, where reasonable, design and build single-span bridges instead of continuous-span bridges.

- Reducing the span lengths, when feasible, and correspondingly reducing the box girder depths, can somewhat reduce the risk of transverse deck cracking. Reducing box depth and increasing web spacing will reduce tensile stresses in the deck.

- To reduce the risk of deck cracking at least a small amount, web spacing can be increased and deck thickness can be increased; structurally, increasing the web spacing will typically require a thicker deck or deeper or wider web, with the costs of the change somewhat offsetting each other.

SOLUTIONS ADOPTED BY OTHER STATE DOT'S

Kansas DOT and University of Kansas

Kansas DOT has completed extensive work on investigating the causes of deck cracking and has implemented specifications aimed to reduce deck cracking (Report No. FHWA-KS-09-*10 Development and construction of low-cracking high-performance concrete bridge decks: construction methods, specifications and resistance to chloride penetration*.) These specification changes arise from research conducted by the University of Kansas for the *Construction of Crack-Free Bridge Decks,* 10-year Transportation Pooled Fund Study, Project No. TPF-5(051). The Kansas DOT is the lead agency for this project. The Federal Highway Administration (FHWA), and DOTs from Delaware, Idaho, Indiana, Michigan, Minnesota, Mississippi, Missouri, Montana, New Hampshire, North Dakota, Oklahoma, South Dakota, Texas, and Wyoming provided funding and representatives to serve on a technical oversight committee for this project. Industry groups also participated in the study. Extensive research has been conducted for this project resulting in numerous technical papers. As part of the study, 20 bridge decks using low-cracking, high performance (LC-HPC) specifications were constructed and compared to conventional bridge decks. The cracking was much less in the decks using the LC-HPC specifications. The results of the comparisons demonstrate that these measures are highly effective in reducing cracking in concrete bridge decks. A summary of the results of this project is provided in *Low-Cracking, High Performance Concrete Bridge Decks*, *Case Studies over First 6 Years*, by Darwin et al. [2010].

Lindquist, Darwin and Browning [2005] measured cracking in fifty-nine steel girder bridges and found correlations between cracking and the volume of cement paste. Cement paste is the fraction of concrete that does not contain fine or coarse aggregate. Stronger concretes and concretes with higher slumps cracked more. They recommended that the paste content be specified below 27 percent to reduce cracking. Crack density was found to decrease with increasing amounts of entrained air, with significant decreases when the air content exceeded 6.0 percent. Kansas now specifies an air entrainment of 8.0 ±1.0 percent for LC-HPC deck concrete. Increasing air entrainment reduces concrete settlement and strengths. They also found that some contractors consistently constructed bridge decks with severe cracking while others consistently produced bridges with low cracking.

Deshpande, Darwin and Browning [2007] evaluated unrestrained free shrinkage of concrete for control of cracking in bridge decks and found that concrete shrinkage decreases with an increase in the aggregate content (and a decrease in the paste content) of the mix. For a given aggregate content, no clear effect of water-cement ratio on shrinkage was observed. In general, use of granite coarse aggregates resulted in lower shrinkage than limestone coarse aggregate. The use of partial volume replacement of portland cement by Class C fly ash without changing the water or aggregate content generally leads to increased shrinkage. The use of partial volume replacement of portland cement by blast furnace slag without changing water or aggregate content can lead to increased early-age shrinkage, although ultimate shrinkage may not be affected. An increase in curing period helps to reduce shrinkage. The use of Type II coarse ground cement results in significantly less shrinkage compared to Type I/II cement, supporting the work by Burrows and others. The use of superplasticizers in concrete appears to increase shrinkage to a certain degree and this could indirectly relate to the relaxation of the slump requirements by AASHTO in the mid-1970's and increased deck cracking.

Lindquist, Darwin and Browning [2008] performed work to develop low-cracking, high-performance concrete (LC-HPC) focused on reducing cracking of decks instead of achieving high strengths or very low permeability. They developed a concrete mixture design program for aggregate optimization, entitled *KUMix*, [2006] which can be obtained from the University of Kansas website [http://www.iri.ku.edu/projects/concrete/phase2.html]. The study also performed free-shrinkage tests to evaluate various concrete mixtures, and evaluated the construction and performance of various bridge decks in Kansas built with the LC-HPC. The free-shrinkage testing included fifty-six concrete batches. Wet curing was typically maintained for the first seven or fourteen days, after which free (unrestrained) drying strains were compared. The free shrinkage in the concrete wet-cured for fourteen days was only slightly less than in those wet-cured for seven days. Contrary to earlier studies by others, the 2008 study did not find that using a high-range water reducer to maintain slump increased shrinkage, although the study maintained a low concrete slump of only 2.25 to 3.5 in. (57 to 89 mm) The study found that the addition of a shrinkage-reducing admixture (SRA) significantly reduced the free shrinkage, cement type did not have a great effect on free shrinkage, and the addition of Class F fly ash increased early-age shrinkage for concrete with either low- or high-absorption aggregate. The early-age effects of silica fume and slag were mixed and depended upon the aggregate used; shrinkage was generally reduced when these concretes were cured fourteen days verses only seven days.

McLeod, Darwin, and Browning [2009] found that the Kansas LC- HPC concrete mixes with lower paste contents tended to have increased chloride permeability. The presence of ground granulated blast furnace slag (ggbfs) and silica fume and longer curing times (from seven to fourteen or twenty-eight days) decreased permeability. Changes in w/c ratio between 0.41 and 0.45 in concrete made with Type I/II cement due to retempering only had a small effect on permeability. Use of coarser ground Type II cement tended to increase permeability compared to a finer Type I/II cement. However, the LC-HPC mixes had lower permeability than the standard Kansas deck mixtures, which had higher paste contents, lower w/c ratios, and lower air contents.

Kansas (McLeod, et. al, 2009) requires covering deck concrete with pre-wetted burlap within ten minutes of finishing; however, most contractors actually require more time to get curing blankets in place. No increase in cracking of field placed decks was noted by Kansas even when up to twenty minutes was used to place the burlap.

Evaluation of the construction of 20 LC-HPC (14 in Kansas and six in other states) (Darwin et al. (2010)) found that successful LC-HPC bridge deck construction resulting in a reduction in deck cracking is repeatable but that clear and consistent communication between the contractor, owner and testing personnel is vital for a successful project. Coarse aggregates were primarily crushed limestone with some local requirements for granite or quartzite. Fine aggregates were predominately river sands, with some being slightly alkali reactive. Except for two LC-HPC decks and one control bridge with prestressed concrete girders, all test bridges have steel girders. Experiences gained during these bridge constructions were used to modify the Kansas bridge deck specifications.

Recommendations Implemented by Kansas for LC-HPC Bridge Decks:

Concrete

- Use concrete with paste content below 27 percent.
- Compressive strength of 3500 psi (24.13 MPa) at 28 days.
- w/c range from 0.43 to 0.45.

- Maximum cement content of 540lbs/CY.
- Air entrainment of 8.0 ±1.0 percent.
- Design slump range from 1.5 to 3 in. (38 to 76 mm), with a maximum allowable of 3.5 in. (89 mm).
- Mixes must conform to optimum grading per a concrete mixture design program for aggregate optimization, entitled KUMix, [2006] [http://www.iri.ku.edu/projects/concrete/phase2.html].
- Provide a qualification batch that consists of a minimum of 6 CY of concrete that meets all of the specifications and is produced at least 35 days prior to placement of a qualification slab.

Construction Practices

Contractor Preparation

- Require mandatory pre-bid conference with contractors to discuss requirements for LC-HPC bridge decks.
- Require contractor to submit a Quality Control plan detailing procedures for controlling evaporation rate.
- Require preconstruction meetings with contractor to discuss cracking prevention.

Qualification Slab

- After the qualification batch has been approved, the construction of a qualification slab is required is constructed 15 to 45 days prior to placing concrete in the bridge deck. The slab must be identical in geometry as the deck, but does not need to be elevated. The methods, equipment, crews and concrete are required to be the same as for the placement of the bridge deck.

Concrete Temperature Control

- At time of placement, limit the temperature of plastic concrete from 50 to 75°F (10 to 24°C).

Evaporation Rate

- Require evaporation rate to be less than 0.2 lb./sq. ft./hr. Measurements must be taken prior to and at least once an hour during placement. If the evaporation rate equals or exceeds 0.2 lb./sq. ft./hr., measures must be taken to reduce the evaporation rate below this value.

Concrete Placement

- Require that concrete be placed by conveyor belt or concrete bucket. Pumping is only allowed if the contractor can show that the approved mix can be successfully pumped, either during the construction of the qualification slab or at least 15 days prior to placing the concrete in the deck.

Consolidation

- Require concrete to be consolidated by use of vertically-mounted internal gang vibrators.

Fogging

- Require continuous machine-mounted fogging of the entire placement width immediately behind finishing operations. The fog spray shall provide a “gloss to semi-gloss water sheen,” but not deposit excess water on the concrete surface.

Finishing

- Do not allow tining of plastic concrete. Screed and finish concrete and begin curing quickly.

Curing

- Curing must begin immediately after finishing and continue uninterrupted for at least 14 days.
- Use of curing compounds is prohibited until after the 14-day curing period.
- One layer of saturated burlap must be placed on the surface of plastic concrete within 10 minutes after concrete strike-off until the end of the 14-day curing period. The burlap must be maintained in a fully wet condition using a self-propelled machine mounted fogging equipment, or other approved methods, until the concrete has set to allow foot traffic. Soaker hoses are then placed to keep the burlap continuously wet. Within 12 hours, white polyethylene film must be placed and secured over the entire structure to form a waterproof cover.

Drying after Curing Period

- After the 14-day curing period and within 30 minutes of removing the burlap and polyethylene film, apply two coats of curing compound while the concrete is still moist. The second coat is to be applied perpendicular to the first coat to provide a uniform coating. The curing compound shall not be disturbed or marred for 7 days.

Grinding and Grooving

- Any required grinding or grooving shall occur after the 14-day curing and 7-day drying period.

Post-Construction Conference

- A post-construction conference is held to discuss successes and problems for the project.

Pennsylvania

In response to severe cracking in significant bridges, the Pennsylvania DOT (Bryan Spangler of Pennsylvania DOT and Paul Tikalsky of the University of Utah) investigated bridge deck cracking. (HPC Bridge Views, Issue No. 45, Fall 2006). Based on its description, some of the cracking observed was likely related to plastic shrinkage cracking due to inadequate early curing (as seen on the recent Caltrans deck placements). Pennsylvania DOT made the following changes to concrete mixtures and construction practices that reportedly have dramatically reduced the frequency and width of cracks (cracking has not been eliminated).

Recommendations implemented by Pennsylvania:

Mix design

- increase concrete w/c from 0.40 to 0.43
- decrease cementitious content from 650 to 588 lb./cu yd. (386 to 349 kg/cu m)
- decrease the percentage of slag from 50 to 42 percent
- decrease the target slump from 6.0 to 4.5 in. (152 to 114 mm)
- reduce maximum allowable concrete temperature at placement from 80 to 75°F (27 to 24°C)

Construction Practices

- place positive moment regions on one day followed by negative moment regions within three days
- apply moist curing immediately and maintain for ten days with pigmented curing compound applied after moist curing is complete
- review and improve quality control and quality assurance operations to control concrete and construction

South Carolina

Hussein (HPC Bridge Views, Issue No. 45, Fall 2006) reported on work in South Carolina to reduce deck cracking. Use of a moderately high strength (HPC), high cementitious content (782 lbs./cu yd.), low w/c (0.37) deck mix that contained silica fume resulted in severe deck cracking. Slow application of wet curing and load-induced cracking were initially suspected as the cause of the cracking. However, the large cement content was later determined to be the major factor in causing the cracking. South Carolina now requires modified mix designs, improved initial curing, and trial mixes. These changes have reduced the incidence of cracking.

Babaei and Purvis [1996] concluded that to limit the average crack spacing to 30 ft. (9m) or more, the twenty-eight day free drying shrinkage measured in the laboratory had to be less than 400 microstrain (less than 700 microstrain, long-term) and the maximum temperature differential between the concrete and the supporting girders must be limited to 22°F (11°C), or the thermal free strain had to be less than 121 microstrain for the first twenty-four hours after placement.

Recommendations implemented by South Carolina:

- modify concrete mix designs by reducing cement content to reduce shrinkage
- improve initial moist curing procedures
- perform trial mixes of bridge deck concretes

Texas and University of Texas at Austin

Folliard [2009] prepared a work plan to develop a predictive model for bridge deck cracking and strength development. This laboratory and field based research is aimed at developing a bridge deck cracking model that can be integrated into ConcreteWorks, a suite of programs developed for TxDOT by this same research team. This project is scheduled to be completed in August 2011. The bridge deck cracking model would account for thermal shrinkage caused by heat of hydration, anticipated temperatures in the field, and deck restraints. The goal for model is also to predict moisture gradients for a range of mixtures subjected to various ambient conditions, and to use the moisture gradients to predict stresses due to drying shrinkage gradients. In addition to thermal and drying shrinkage, the model is to account for plastic and autogenous shrinkage.

Washington State

Washington State has seen early-age cracking in many concrete bridge decks. Oiao, et. al. [2010] and Zhuang [2009] studied concrete mixes used in Washington and found that adding fly ash caused concrete to crack sooner. Mixes with lower paste volume had a lower tendency for cracking and both size and source of coarse aggregates play a very important role in the concrete properties. The use of SRAs significantly reduced the free and restrained shrinkage. Recommendations include using a SRA, do not use fly ash, design concrete with lower paste volumes, use largest practical coarse aggregate size, and trial batching is valuable when using several cementitious materials and chemical admixtures in the same concrete batch. Khaleghi [2001] reports the HPC decks in Washington use fly ash concrete but that two coats of curing compound are required followed by continuous wet curing for 14 days. Pre-construction conferences are required five to ten days before deck placement with the contractor. Changes have been recommended to the Class D deck concrete including a maximum coulomb value of 1,500 C (AASHTO T277) and a maximum shrinkage of 350 microstrain at 28 days (AASHTO T160). These recommendations are being evaluated on trial projects.

Research by Other DOTs

Many DOTs have conducted their own research or studies on the causes and mitigation of early-age deck cracking in the last 10 years or so. A list of some of the DOTs and studies are provided below. These studies typically suggest general changes to mix design and construction practices, rather than specific changes to current specifications.

- Alabama DOT (2010) Evaluation of Cracking of the U.S. 331 Bridge Deck, Schindler, A., Hughes, M., Barnes, R., & Byard, B.
- Arkansas DOT (2006) The Effect of Mixture Performance on Bridge Deck Performance, Sanders, C.L., University of Arkansas
- Colorado DOT (2003) Assessment of the Cracking Problem in Newly Constructed Bridge Decks in Colorado, Xi, Y., Shing, B., Abu-Hejleh, N., Asiz, A., Xie, Z., & Ababneh, A.
- Idaho DOT (2008) Synthesis into Causes of Concrete Bridge Deck Cracking and Observation on the Initial Use of High Performance Concrete I the U.S. 95 Bridge over the South Fork of the Palouse River, Schemeckpeper, E.R., & Lecoultre, S. T., University of Idaho
- Illinois DOT (2003) High Performance Concrete for Transportation Structures, Lange, D. A., Roesler, J. R., D'Ambrosia, M. D., Grasley, Z. C., Lee, C. J., & Cowen, D. R.
- Indiana DOT (2003) Investigation of Bridge Deck Cracking in Various Bridge Superstructures Systems, Frosch, R. J., Blackman, D. T., & Radabaugh, R. D.
- Michigan DOT (2003) Investigate Causes & Develop Methods to Minimize Early-Age Deck Cracking on Michigan Bridge Decks, Aktan, H., Fu, G., Dekelbab, W., & Attanayaka, U.
- Minnesota DOT (1999) Transverse Cracking in Bridge Decks: Summary Report, French, C. E., Le, Q. T. C., Eppers, L. J., & Hajjar, J. F.
- Mississippi DOT (2010) Shrinkage and Durability Study of Bridge Deck Concrete, Varner, R. L.
- Missouri DOT (2003) Laboratory Testing of Bridge Deck Mixes, Missouri Department of Transportation

- New Jersey DOT (2002) Cause and Control of Transverse Cracking in Concrete Bridge Decks, Saadeghvaziri, M. A., & Hadidi, R.
- New Jersey DOT (2010) Bridge Deck Cracking and Composite Action Analyses, Nassif, H., Suksawang, N., Najm, H., & Lewis, R.
- New Mexico DOT (2008) Bridge Deck Fogging System: Evaluation of Field Implementation of Fogging System used During Construction of Bridge Deck Construction, New Mexico State University.
- New York DOT (2007) NYSDOT Bridge Deck Task Force Evaluation of Bridge Deck Cracking on NYSDOT Bridges, Curtis, R., & White, H.
- Ohio DOT (2006) Transverse Cracking of High Performance Bridge Decks After One Season or 6 to 8 Months, Miller, R., Mirmiran, A., Ganesh, P., & Sappro, M.
- Oregon DOT (not yet completed) Development of Shrinkage Testing Protocols and Limits for ODOT High Performance Concrete, Ideker, J., Oregon State University
- Texas DOT (2001) Restrained Shrinkage Cracking of Concrete Bridge Decks: State-of-Art Review, Brown, M., Sellers, G., Folliard, K., & Fowler, D.
- Washington State DOT (2009) Evaluation of Concrete Mix Designs to Mitigate Early-Age Shrinkage Cracking in Bridge Decks, Zhuang, J., Washington State University
- Wisconsin DOT (2010) Concrete Cracking in New Bridge Decks and Overlays, Wan, B., Foley, C., & Komp, J., Marquette University

FINDINGS - RECOMMENDATIONS TO CALTRANS

Factors affecting bridge deck cracking are complex and multiple. The recommendations provided in this section will reduce the risk or severity of cracking in almost all bridge decks, with specific emphasis on the box-type bridges built by Caltrans. For most bridge decks, cracking will depend primarily upon the early history of bridge temperatures, concrete properties, and the environment. Interaction of the deck with its supporting elements also affects the cracking risk, but typically to a lesser extent on Caltrans box structures except near continuous span supports. During the first several days, large temperature differences and corresponding stresses can develop, and excessive drying can cause shrinkage stresses, which can cause microdamage or visible cracking.

The many factors that cause early-age cracking in bridge decks were researched by performing a review of the literature, field instrumentation and analytical studies. Many previous studies have identified potential solutions to reduce the risk of cracking. In general, reducing stress in the bridge deck is a key focus to reduce deck cracking. This can be done by reducing concrete autogenous, drying and thermal shrinkage, reducing deck restraint, reducing deck curvature, or lowering concrete modulus and increasing concrete creep.

A Review Panel of experts in research, design, and construction of bridge decks was formed to review the recommendations on this report. The Review Panel included the following experts:

- Gary Janco of C.C. Meyers Construction, University of Illinois at Urbana - Champaign
- John Bolander, University of California at Davis
- Boris Stein, Twining, Inc.
- Mohammed Fatemi, Alta Vista Solutions
- David Darwin, Kansas University
- David Lange, University of Illinois, UIUC

These experts were given two opportunities to review and comment on the recommendations of this study.

Caltrans Specifications

Caltrans Specifications - Section 90 Portland Cement Concrete (Issued 11-30-10)

Concrete used for bridge decks is required to have a minimum cementitious content of 675 lbs./cu. yd., and a minimum 28-day compressive strength of 3,600 psi (24.82 MPa) unless otherwise specified. The amount of free water is limited to 310 pounds per cubic yard, plus 20 pounds for each required 100 pounds of cementitious material in excess of 550 pounds per cubic yard based on the minimum cementitious material. A maximum shrinkage specification of 0.045 percent at twenty-eight days is specified for deck concrete, and cement must conform to ASTM C150 Type II or Type V. For Type II cement, the C_3S content shall not exceed 65 percent. Blended cements (AASHTO M 240) are allowed without limits on the pozzolans content. Blended cement shall be comprised of Type II or Type V cement and supplemental cementitious material (SCM). Cement alkali is limited to 0.60 percent and the autoclave expansion shall not exceed 0.50 percent.

Fly ash is limited to AASHTO M 295 Class F with additional restrictions. Ultra-fine fly ash (UFFA), raw or calcined natural pozzolans, metakaolin, ground granulated blast furnace slag (ggbfs), and silica fume are also SCM's allowed with specific requirements. Supplementary cementitious materials are required in all deck concrete, except for precast concrete using innocuous aggregates. Aggregate quality and gradations limits are specified. Coarse aggregate limits include 1½-inch by ¾-inch (38 mm by 19mm) maximum size but smaller sizes are allowed. Concrete slump is normally limited to a penetration of 2½ to 3 inches (slump of 5 to 6 inches [127 mm to 152 mm]); however, Section 90 allows if Type F or Type G chemical admixtures are added to the mix, the penetration requirements shall not apply and the slump shall not exceed 9 inches after the chemical admixtures are added.

In summary, for bridge decks Caltrans typically specifies and uses concrete with a large cementitious content, moderate aggregate size, moderate to high paste content, moderately high slump, and fly ash or other SCM. These properties or components may tend to aggravate deck cracking. When modifying these properties to reduce the risk of cracking, one needs to consider possible adverse effects on other concrete properties such as strength gain and long-term durability related mainly to alkali-aggregate reactions.

Caltrans Deck Construction Specifications

WJE reviewed two documents that provide recommendations for the curing of bridge concrete decks. Some of the more relevant Caltrans provisions related to deck curing are as follow:

Caltrans 2006 Standard Specifications:

General Provisions:

Top surface of highway bridge decks should be cured by both the curing compound method and the water method. The concrete shall be kept continuously wet by the application of water for a minimum curing period of 7 days after the concrete has been placed.

Curing Compound Provisions:

Surfaces of the concrete that are exposed to the air shall be sprayed uniformly with a curing compound. Curing compound shall be applied at a nominal rate of one gallon per 150 square feet, unless otherwise specified. Curing compound shall be applied using power operated spray equipment.

The curing compound shall be applied to the concrete following the surface finishing operation, immediately before the moisture sheen disappears from the surface, but before any drying shrinkage or craze cracks begin to appear. In the event of any drying or cracking of the surface, application of water with an atomizing nozzle as specified in Section 90-7.01A, "Water Method," shall be started immediately and shall be continued until application of the compound is resumed or started; however, the compound shall not be applied over any resulting freestanding water. Should the film of compound be damaged from any cause before the expiration of 7 days after the concrete is placed in the case of structures and 72 hours in the case of pavement, the damaged portion shall be repaired immediately with additional compound.

Water Curing Provisions:

When a curing medium consisting of cotton mats, rugs, carpets, or earth or sand blankets is to be used to retain the moisture, the entire surface of the concrete shall be kept damp by applying water with a nozzle that so atomizes the flow that a mist and not a spray is formed, until the surface of the concrete is covered with the curing medium. The moisture from the nozzle shall not be applied under pressure directly upon the concrete and shall not be allowed to accumulate on the concrete in a quantity sufficient to cause a flow or wash the surface. At the expiration of the curing period, the concrete surfaces shall be cleared of all curing mediums.

At the option of the Contractor, a curing medium consisting of white opaque polyethylene sheeting extruded onto burlap may be used to cure concrete structures. The polyethylene sheeting shall have a minimum thickness of 4 mil and shall be extruded onto 10-ounce burlap.

If the Contractor chooses to use polyethylene sheeting or polyethylene sheeting on burlap as a curing medium as specified above, these mediums and any joints therein shall be secured as necessary to provide moisture retention and shall be within 3 inches of the concrete at all points along the surface being cured. When these mediums are used, the temperature of the concrete shall be monitored during curing. If the temperature of the concrete cannot be maintained below 140°F (60°C), this method of curing shall be discontinued, and one of the other curing methods allowed for the concrete shall be used.

When concrete bridge decks and flat slabs are to be cured without the use of a curing medium, the entire surface of the bridge deck or slab shall be kept damp by the application of water with an atomizing nozzle as specified in the preceding paragraph, until the concrete has set, after which the entire surface of the concrete shall be sprinkled continuously with water for a period of not less than 7 days.

Caltrans 2001 Bridge Deck Construction Manual:

General Provisions:

The general provisions refer to section 90-7.03 of the 1988 Standard Specifications, "Curing Structures", "The top surface of Highway bridge decks shall be cured by both the curing compound method and the water method, except that the curing compound shall be the ...pigmented curing compound...." In most areas of the State and in all the major metropolitan areas, water based pigmented curing compound is the only type of curing compound permitted by the Air Quality Board.

Curing Compound Provisions:

Curing compound shall be applied progressively during deck finishing operations immediately after finishing operations are completed on each individual portion of the deck. The water cure shall be applied not later than 4 hours after completion of deck finishing, or for portions of the deck completed after normal working hours, the water cure shall be applied not later than the following morning. Be sure that the curing compound is sufficiently dry and the concrete has sufficiently set before the rugs or mats are placed on the deck.

Fogging Provisions:

During hot weather especially if it is windy and/or the humidity is low, fogging the fresh concrete deck may be necessary if the cure application is late. The contractor must begin fogging the deck immediately before the surface sheen of the concrete disappears and before surface cracks begin to appear. The purpose of fogging is to keep the concrete cool and prevent premature moisture loss and uneven shrinkage in the concrete before the cure is applied. Fogging can be detrimental to the deck if too much water is applied and it puddles or runs off the deck and washes the fresh concrete. Fogging must also be done with the correct equipment - a fogging nozzle in good operating condition that adequately atomizes the water. See Bridge Construction Records and Procedures Memo 105-3 and 105-4 for additional information.

Water Curing Provisions:

The water cure shall be applied not later than 4 hours after completion of deck finishing, or for portions of the deck completed after normal working hours, the water cure shall be applied not later than the following morning. The concrete shall be kept wet continuously for seven days. Cotton mats, rugs, carpets, burlene or earth or sand blankets may be used as the moisture retaining medium. Be sure that the curing compound is sufficiently dry before the rugs or mats are placed. It is recommended that the contractor wet down the deck before the rugs or mats are placed. The moisture retaining medium must be wetted immediately after placement and kept wet for at least 7 days.

In summary, Caltrans requires the use of both a curing compound and wet curing on bridge decks, however, wet curing must be delayed until the curing compound dries and can be delayed until the next morning (as was done on the Olive Lane Bridge). The use of fogging is also contemplated; however, there is not a clear temperature or evaporation rate threshold established for when fogging is necessary. The contractors generally followed the specified curing procedures on the two decks investigated as part of this study, however they did not use all of the allowable provisions within the specifications to avoid the plastic cracking that occurred. The curing compound application is delayed until immediately before the moisture sheen disappears from the surface, but before any drying shrinkage or craze cracks begin to appear. Further, wet curing is delayed until the curing compound dries. The specifications warn the contractor of the dangers of too much curing water too early, likely to the detriment of cracking. Significant revisions to the Caltrans curing specifications are needed to avoid deck cracking.

Primary Recommendations

Recommendations have been divided into Primary Recommendations that are considered to be the most important and effective and Secondary Recommendations that are beneficial but may be less effective or important. This section includes our Primary Recommendations to Caltrans.

Cementitious and Paste Content (Concrete)

Caltrans specifications currently require concrete used for bridge decks to have a minimum cementitious content of 675 pcy, which is excessive for most bridge decks. Reducing the high to very-high cementitious content of some bridge deck mixes used by Caltrans should be a primary goal to reduce deck cracking. Doing so should also produce more economical concrete mixes.

- Lindquist, Darwin and Browning [2005] measured cracking in fifty-nine steel girder bridges and found correlations between cracking and the volume of cement paste. They determined that stronger concretes and concretes with higher slumps cracked more. They recommended specifying a paste content below 27 percent to reduce cracking. Kansas requires concrete with paste content below 27 percent and a maximum cement content of 540 pcy but goes as low as 500 pcy.

- Pennsylvania has decreased cementitious content of bridge deck concrete from 650 to 588 pcy which along with other changes dramatically reduced deck cracking.

- Krauss and Rogalla [1996] tested restrained concrete ring samples and found that concrete shrinkage and cracking increased with increasing paste content. The cement paste is the component of the concrete that shrinks, so reducing this volume reduces shrinkage and cracking.

- For this Caltrans project, thermal analyses of a range of concrete box sections under different environmental conditions indicate that reducing early temperature gains from hydration typically reduces the risk of early deck cracking more than any other factor. Reducing the hydration heat significantly reduces peak temperatures from which the deck must cool, as well as temperature differences. Temperature analysis performed on the Olive Lane bridge deck placement showed that a decrease in the cement content from 675 pcy to 550 pcy reduced the estimated peak concrete temperature by 22°F (11°C) and the maximum temperature differential in the bridge by 13 to 23°F (6 to 11°C).

In lieu of specifying *minimum* cement content, Caltrans could specify *maximum* paste and cement contents for bridge deck concrete. This will reduce concrete shrinkage and heat of hydration stresses. However, Review Panel members generally do not support specifying a somewhat arbitrary minimum cement content mainly due to concerns with workability, and they prefer to limit strength instead (Which could also be problematic under job conditions. What do you do when the concrete tests too strong at 28 days after the placement?). They did, however, agree that the minimum cementitious content requirement should be eliminated to allow for leaner concrete mixes. Panel members suggested specifying concrete strengths at 56 days instead of 28 days but were mixed on specifying a maximum concrete strength. The lowest possible cement content should be used to minimize the risk of cracking by reducing shrinkage, initial hydration temperatures, and thermal stresses.

Recommendations

- Eliminate the minimum cementitious content requirement to allow leaner mixes
- Specify a maximum cementitious content of 600 pcy.
- Specify maximum paste content of 27 percent. Consideration of supplementary materials necessary to combat ASR might affect this limit.

Curing Methods and Plastic Cracking (Construction)

Severe plastic cracking developed in both Caltrans bridge decks monitored for this study. Plastic shrinkage cracks are those that occur while the concrete is relatively fresh and has not fully hardened. They usually appear on exposed unformed surfaces, and can occur anytime that ambient conditions (temperature, humidity, and wind velocity) are conducive to rapid evaporation. The plastic shrinkage cracking that developed is attributable to the environmental conditions and the Contractor's placing and initial curing procedures. Contractor procedures essentially followed project specifications but were not sufficient to prevent cracking. Caltrans Specifications - Section 90 Portland Cement Concrete (issued 11-30-10) requires application of a membrane forming curing compound followed by continuous moist curing for seven days minimum using cotton mates, rugs, carpets or earth or sand blankets. However, commencement of moist curing can be delayed overnight. Curing techniques and curing duration have crucial effects on the strength and durability of concrete as well.

Plastic cracking of concrete is discussed in the Caltrans Bridge Construction Records and Procedures manual (Memo 105-4.0). [Procedures, December 24, 2010] The attachment to Memo 105-4.0 is used to determine when precautionary measures are needed. Evaporation rates of 0.2 psf/hr. or greater require precautionary measures while the risk of cracking is still present at rates between 0.1 and 0.2 psf/hr.

Plastic Shrinkage Cracking

The following information relates to curing practices that primarily affect plastic shrinkage cracking occurring within the first 24 hours of placement:

- Huo and Wong [2005] performed a study to examine the early-age behavior of high-performance concrete (HPC) under various curing methods. Laboratory experiments were conducted to investigate the early-age shrinkage development, temperature change, and evaporation rate when different curing methods including burlap, cotton mat, polyurethane blanket, and curing compound were used. The results show that curing methods that allow evaporation (e.g. cotton mat or burlap) minimize hydration heat, while a combination of wet curing and moisture barrier (polyurethane blanket) minimizes the final shrinkage.
- Kansas requires concrete to be fogged using continuous machine-mounted fogging of the entire placement width immediately behind finishing operations. The fog spray shall provide a "gloss to semi-gloss water sheen," but not deposit excess water on the concrete surface. They do not allow tining of plastic concrete. Screed and finish concrete and begin curing quickly. Curing must begin immediately after finishing and continue uninterrupted for at least 14 days. A curing compound is prohibited until after the 14-day curing period. They require one layer of saturated burlap must be placed on the surface of plastic concrete within 10 minutes after concrete strike-off. However, contractors actually often take up to 20 minutes to complete this task. The burlap must be maintained in a fully wet condition using a self-propelled machine mounted fogging equipment,

or other approved methods, until the concrete has set to allow foot traffic. Soaker hoses are then placed to keep the burlap continuously wet. Within 12 hours, white polyethylene film must be placed and secured over the entire structure to form a waterproof cover. After the 14-day curing period and within 30 minutes of removing the burlap and polyethylene film, two coats of curing compound are applied. The second coat is to be applied perpendicular to the first coat to provide a uniform coating. The curing compound shall not be disturbed or marred for 7 days. Any required grinding or grooving shall occur after the 14-day curing and 7-day drying period.

- Pennsylvania requires immediate moist curing be maintained for ten days with pigmented curing compound applied after moist curing is complete

The most effective means of avoiding the excessive loss of bleed water (reducing evaporation) and plastic drying shrinkage cracking is to use fogging during construction, followed by rapid placement of wet curing. The fog mist should be applied to the concrete surface from the upwind side of the work, and done with a commercial-grade fog nozzle that provides broad coverage and produces a mist so fine that it is nearly impossible to damage the concrete surface. The contractor should not work the fogged water into the concrete surface during finishing. Increasing the bleeding capacity of the concrete may reduce the risk of plastic cracking but is usually not practical; although use of a water-reducing admixture containing hydroxylated carboxylic acid tends to increase bleeding.

Rapid and continuous deck placement operations typically produce the highest quality decks. Immediate and smooth finishing operations allow curing to be applied more rapidly, reducing the risk of plastic shrinkage cracking. Placing of prewetted burlap can be cumbersome and difficult for some contractors, but it is the most common and effective means to prevent plastic cracking if done timely. Some contractors mount rolls of burlap on the finishing machine to automatically cover the fresh concrete. Concrete tining should be prohibited as this slows the application of wet curing; instead, grooving should be done for skid resistance after the concrete is fully cured.

Review Panel members generally agreed with recommendations to avoid plastic shrinkage cracking by using an evaporation retarder, applying water fogging until wet curing media is installed, applying wet curing media as soon as practical (10 to 20 minutes after finishing), pre-moisten forms and reinforcing, and avoiding batching dry aggregates. Erecting sun shades or wind breaks were thought to be good ideas but more difficult to implement. It is important that evaporation retarders and fogging water is not worked into the concrete surface (only apply after floating and finishing is complete). It was felt that some of these items are already in the specifications and that Caltrans needs better enforcement in some cases. The Panel did not think that requiring a monetary penalty to Contractors if plastic cracking occurs was a good idea.

Later-age Deck Cracking

The following discussion addresses curing effects on later-age cracking and concrete durability:

- Krauss and Rogalla [1996], Altoubat and Lange [2001] and others have found that concrete cured for extended periods (even up to 60 days) typically cracked at similar or shorter periods of drying in restrained ring tests. Longer curing increases concrete stiffness and lessens creep so that higher stresses develop upon subsequent drying.

- Altoubat and Lange [2001] found that rewetting the concrete relaxed stresses at a high rate within twenty-four hours of water application, and shrinkage stress from redrying occurred at a lower rate than initial drying. Thus a curing program of periodic wetting to keep the stress buildup from exceeding 50 percent of the concrete tensile strength at all times may prevent or reduce early-age microcracking and ultimate cracking.

- Deshpande, Darwin and Browning [2007] found that increasing curing from 7 to 14 days can help in controlling shrinkage and the need for good moist curing to produce a dense and durable concrete surface is an important consideration to balance.

- The difference in the surface temperatures and the interior concrete temperatures has been used to predict cracking in concrete structures. In plain walls, Freiesleben-Hansen and Pedersen [RILEM 42 CEA, 1981] suggest that the difference in surface and interior temperatures be limited to 68°F (34°C) and temperatures between new and existing walls be limited to 55 to 60°F (27 to 30°C) to limit cracking. Thermal blankets installed on the Olive Lane Bridge after the peak temperature due to heat of hydration (at 22 hours) reduced the diurnal temperature changes and strains in the deck and kept the deck warmer over the curing period.

- A second application of white pigmented curing compound after wet curing was complete on the Olive Lane Bridge reduced peak diurnal temperatures by about 4°F (2°C) compared to the single coat applied after finishing but before wet curing.

A membrane-forming curing compound applied immediately after finishing is used on many Caltrans job to reduce initial concrete drying. However, this allows the Contractor to delay wet curing and it has not been effective in preventing plastic shrinkage cracking (as was observed at the two bridges that were monitored for this study). The intent of the membrane curing compound is to form a very thin layer of polymer (plastic) to inhibit loss of water from the concrete. However, when applied immediately to fresh concrete, the concrete bleed water continues to rise displacing the curing compound solids and results in a discontinuous and somewhat ineffective membrane coating. The most effective means of avoiding the loss of bleed water (reducing evaporation) is fogging during construction, followed by rapid placement of wet curing. Caltrans curing specifications should be modified to require immediate misting and wet curing and to prohibit the use of membrane curing compound until after wet curing is complete.

After wet curing is complete, moisture leaves concrete slowly due to the slow migration of moisture toward a dry surface to equalize dampness. The humidity in the surrounding air must be reduced somewhat below 100 percent before any moisture leaves concrete, because the water in concrete is not pure. Curing compounds or barrier type sealers can slow the rate of drying after wet curing is complete and reduce deck stresses. Applying curing compound to a hard, dry surface after the wet curing is completed provides a more continuous and effective layer of curing compound then when placed directly on plastic concrete that is still bleeding. Apply curing compounds uniformly in two perpendicular coats to affect complete coverage.

Most bridges built by Caltrans are of the concrete box girder type, where the space immediately below the deck is enclosed by the box webs and bottom flange, essentially creating an insulating air space through which very little transfer of heat (cooling) occurs from the deck. As such, decks on box bridges cool primarily from the top side, whereas decks built between steel or concrete girders typically cool from both sides and much more rapidly. Therefore, especially with box girder bridges, it is important to minimize early temperature gain and facilitate cooling to reduce cracking risk. Immediate wet curing will help keep initial hydration temperatures down by direct cooling and evaporation.

Curing blankets insulate the deck from the environment and kept the deck warmer during the curing period. It is important not to apply blankets too early as there is a risk of trapping heat into the deck that will increase early temperatures and stress differentials. However, for decks cast in the early morning on hot summer days, the blankets may actually shield the concrete from the sun and keep temperatures lower; further research is needed on when it is best to place blankets depending on site conditions. To reduce solar heat gain, which can be especially significant in a deck of a box girder bridge (because of limited cooling from the soffit), burlap should be constantly wet allowing for evaporative cooling or be covered with plastic that is white or light in color to reflect most of the solar radiation. Black or clear plastic should not be allowed. Early wet curing and reflective coverings will reduce early temperatures, early temperature differentials, and both early and later thermal stresses (affected by early residual thermal stresses).

At the conclusion of wet curing, only white-pigmented curing compounds or sealers should be used that will help reduce surface temperatures and deck stresses. White-pigmented curing compounds will reduce surface temperatures during the day by reflecting solar radiation. Caltrans Translab measured a 10°F (5°C) reduction in concrete temperatures when white pigmented curing compounds were applied compared to clear curing compounds and the Olive Lane Bridge noted a 4°F (2°C) reduction of temperatures within the concrete by a second application of curing compound. All Review Panel members felt that applying a white pigmented curing compound after completing wet curing is a good idea.

When concrete contains fly ash, inadequate curing can increase free shrinkage, and extending curing of these concretes should reduce the risk of cracking. Panel members agreed that increasing curing to 14 days would be beneficial, but that it may not be necessarily practical for all projects. They felt curing fly ash concrete for 21 days may be problematic on some jobs.

Summary

Caltrans specifications should allow Contractors some flexibility in curing practices, but hold Contractors accountable for plastic drying shrinkage cracks, which are preventable with good construction practice. The Review Panel members were mixed on hove to hold Contractor accountable for plastic shrinkage cracks. Arguments included that it would be difficult to enforce administratively, would increase contractor risk and thereby costs to Caltrans, and that “concrete cracks” [cracking is inevitable]. However, since plastic shrinkage cracks can have a substantial adverse effect on deck performance and repair of deck cracks may not be fully effective, Contractors should be motivated to avoid cracking in the first place. Performance-based specifications should be applicable since preventing plastic shrinkage cracks is within full control of the Contractor by using proper and well-established curing methods. However, this does not apply to cracking that occurs after the first 24 hours of placement (non-plastic deck cracks).

Recommendations:

- Specify immediate misting and wet curing (cotton mats or prewetted burlap) of finished concrete and prohibit use of membrane curing compound. If plastic is used over the burlap of fabric it should be opaque white or light-colored to reduce solar heat gain. Allowing early evaporative cooling may be beneficial to reduce concrete temperatures.

- Ensure adequate equipment is available to lift and place heavy pre-wetted burlap without damage to the surface. Consider screed mounted rolls of burlap. Dry cotton sheets can be applied to the fresh deck surface and immediately wetted (Illinois DOT). Keep curing media saturated.

- Wet cure deck concrete for 14 days.

- Apply two perpendicular coats of white pigmented membrane forming curing compound after wet curing is complete.

- Hold a pre-job conference to discuss curing and cracking issues with the contractor. Review Caltrans Bridge Construction Records and Procedures Manual (Memo 105-4.0) on preventing plastic shrinkage cracking with the contractor and project staff prior to deck placement.

Research Ideas:

- Rewet the deck after 7 and 14 days of drying (after the initial wet curing) to recover some of the drying shrinkage and relax some of the irreversible drying shrinkage strain. Further field research on this topic is worthwhile.

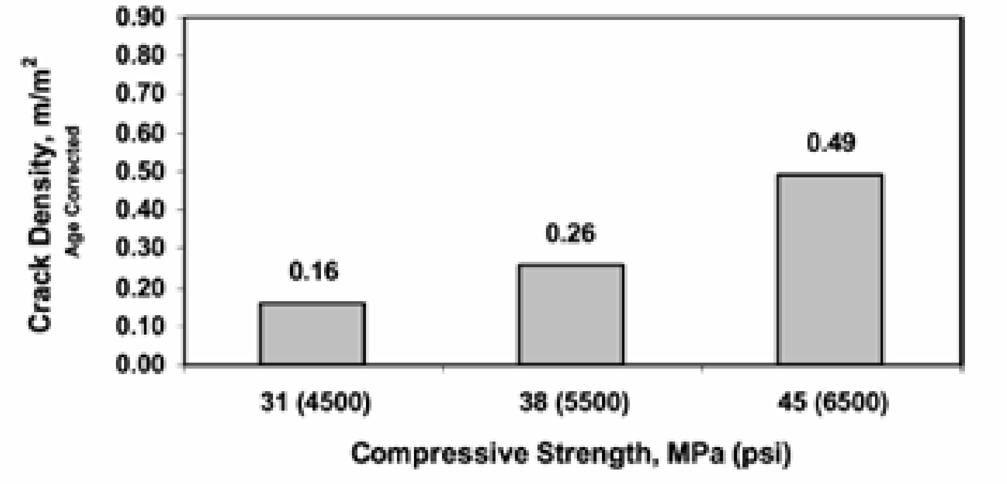
- Evaluate the optimum time to apply and remove insulation blankets on the deck to minimize deck stresses and cracking. When insulation will be used, analytical study should be made before construction to determine the best time to begin insulating the deck and the amount of insulation to be used; for most cases, we suspect that insulation should not be applied until after the concrete has reached its peak temperature and its temperature is decreasing.

Compressive Strength (Concrete)

In bridge decks, higher strength concretes are generally more prone to cracking. High strength concretes are stiffer (have a higher elastic modulus) and thereby develop higher stresses for a given temperature change or shrinkage amount. Additionally, the creep potential that is beneficial at mitigating early temperature stresses and shrinkage stresses declines quickly with increasing strength, so that larger thermal and shrinkage stresses are locked into the deck, which then can increase the risk of deck cracking. Higher strength concretes also typically contain more cement, which increases shrinkage and temperatures during early hydration. Tensile strength, that resists cracking, does not increase enough compared to compressive strength and effective modulus to offset the increased risk of cracking.

Caltrans specifies a minimum 28-day compressive strength of 3,600 psi (25 MPa) for bridge decks unless otherwise specified, which is appropriate. The specifications, however, do not limit the maximum strength, leaving contractors free to use high-strength concrete that may be easier to place but more prone to cracking. The concrete used in the two bridge decks investigated in this study had 28-day compressive strength in excess of 5,000 psi (34 MPa).

- Kansas and many other researchers have shown that increased strength results in increased cracking. Lindquist, Darwin and Browning [2005] measured cracking in fifty-nine steel girder bridges and found correlations between cracking and the concrete strength with crack densities of 0.16 m/sq. m for 4,500 psi (31 MPa) concrete, 0.26 m/sq. m for 5,500 psi (38 MPa) concrete, and 0.49 m/sq. m for 6,500 psi (45 MPa) concrete as shown in the following figure. Stronger concretes and concretes with higher slumps cracked more.



- Krauss and Rogalla [1996] found that higher strength concretes in restrained ring tests tended to crack sooner and more severely. Deck cracking was reported to have increased in severity in the mid 1970's and this coincided with AASHTO changes in deck concrete strength from 3,000 psi (21 MPa) to 4,500 psi (31 MPa) for air-entrained concrete.

- Studies for this Caltrans project also reveal that a higher strength concrete develops greater early temperature and early temperature differentials, primarily because of the larger cement content required to achieve the higher strength. Furthermore, because higher strength concrete typically has a larger modulus of elasticity (is stiffer), it will develop larger thermal and shrinkage stresses for a given amount of cooling or drying, as the underlying bridge structure those movements. Higher strength concretes also tends to creep less than lower strength concrete, reducing the typical stress relieving benefits of creep.

The risk of cracking may be reduced by selecting a concrete mix with low shrinkage that does not excessively exceed the required compressive strength. Per the Review Panel suggestion, specifying concrete strengths at later ages, say 56 days, allows for lower strength concrete that still meets design requirements, assuming construction constraints allow for slower strength gain. Several panel members resisted specification of a maximum concrete strength.

Recommendations:

- Specify a minimum compressive strength for deck concrete of 3,600 psi (25 MPa) at 56 days, unless otherwise required.

- While the Review Panel had mixed replies, specifying a maximum compressive strength of 4,500 psi (31 MPa) at 7 or 14 days should be considered.

Fly Ash and SCM's (Concrete)

Fly ash is limited by Caltrans to AASHTO M 295 Class F with additional restrictions on chemistry. Ultra-fine fly ash (UFFA), raw or calcined natural pozzolans, metakaolin, ground granulated blast furnace slag (ggbfs), and silica fume are also SCM's allowed with specific requirements. Supplementary cementitious materials are required in all deck concrete, except for precast concrete using innocuous aggregates.

- Jensen and Hansen [1996] measured an increase in autogenous deformation of 1000 microstrain after two weeks for concrete containing 10% silica fume. This compared to only 200 microstrain after two weeks for a change in water/cement ratio from 0.40 to 0.25 for a concrete without silica fume. For comparison, the total drying shrinkage of typical concrete over several years is often less than 800 to 1000 microstain. Many other studies have also shown an increase in cracking with silica fume.

- Darwin, D., Browning, J., & Lindquist, W. D. [2004, December] and Krauss and Rogalla [1996] found the silica fume increased concrete strength and restrained ring samples cracked 30% sooner than controls. Recommendations included that silica fume should not be allowed in concrete used for decks due to the increased risk for cracking.

- Lindquist, Darwin and Browning [2008] found that the addition of Class F fly ash increased early-age shrinkage for concrete with either low- or high-absorption aggregate. The early-age effects of silica fume and slag were mixed and depended upon the aggregate used; shrinkage was generally reduced when these concretes were cured fourteen days verses only seven days.

Review Panel members were split on allowing the use of silica fume, but the arguments for allowing silica fume had more to do with issues besides cracking. While silica fume will reduce concrete permeability, very low permeability is not required on most decks in California. Silica fume is not commonly used in Caltrans decks but allowing its use will likely increase both plastic and early-age cracking, which would likely be more detrimental than the benefit of decreased permeability.

Caltrans requires low-calcium fly ash to address a serious need to minimize the deleterious reactions of the alkali-reactive aggregates common throughout the state and somewhat unique to the western states. Therefore, eliminating fly ash to reduce the risk of deck cracking is not practical for Caltrans. Extended wet curing reduces the rate of concrete drying shrinkage and beneficially reduces permeability of fly ash concretes. For concretes containing fly ash, wet curing should extend to twenty-one days to reduce the rate of drying and shrinkage. If it is ascertained that non-reactive aggregates will be used or that ASR reaction can be mitigated in another manner, eliminating the fly ash from the concrete mix and wet curing for fourteen days should reduce the risk of deck cracking. Review Panel members did not agree with eliminating the use of fly ash since it is prescriptive and against “Green” policies. For Caltrans mixes, it is best to require extended wet curing and to test the combination of local materials to be used to determine the effects of the various supplementary cementitious materials on free shrinkage and cracking tendency.

For mixes containing Ultra-fine fly ash (UFFA), raw or calcined natural pozzolans, metakaolin, ground granulated blast furnace slag (ggbfs), and silica fume, testing for unrestrained shrinkage and cracking tendency testing relative to a standard Caltrans control concrete is a good way to evaluate their effect. Without this testing there is a risk that the SCMs will increase deck cracking.

Recommendations:

- Do not allow silica fume in deck concrete.

- Increase the wet curing period for blended cement concrete or fly ash containing concrete to a minimum of 21 days, when able.

- Allow ultra-fine fly ash, raw or calcined natural pozzolans, metakaolin, or ground granulated blast furnace slag (ggbfs) in deck concrete only after testing for unrestrained shrinkage

(AASHTO T160) and cracking tendency (AASHTO T334) is performed relative to a standard control concretes and no increase in shrinkage or cracking tendency is shown.

Air Content (Concrete)

Air entraining admixtures are used to protect concrete from cyclic freezing damage and Caltrans only requires air-entrainment for concrete that will be in a freezing environment. However, in addition to improving durability in a freeze-thaw environment, air entrainment reduces the risk of deck cracking as it slightly lowers the concrete strength and modulus of elasticity of the concrete, improves workability, and reduces concrete settlement.

- The ultimate stain capacity of concrete is about 20 percent higher when concrete is air entrained [Springenschmid and Breitenbucher, 1998].
- Breitenbucher and Mangold [1994] found adding air entrainment to achieve an air content of approximately 3 to 6 percent by volume reduced the cracking temperature (the cooled concrete temperature after hydration at which cracks developed) by about 9°F (4°C).
- Lindquist, Darwin and Browning [2008] found that crack density of bridge decks decreased with increasing amounts of entrained air, with significant decreases occurring when the air content exceeded 6 percent. Kansas now specifies an air entrainment of 8.0 ±1.0 percent for LC-HPC deck concrete. Increasing air entrainment also reduces the risk of concrete settlement cracks.

Review Panel members generally agree with requiring air entrainment for all deck concrete as it will increase workability without increasing shrinkage. The main concern is for the loss in compressive strength when air is used and how that might impact contractors that have mixes that just meet minimum compressive strength requirements. The compressive strength of the deck concrete used in the two bridges studied in this project were well in excess of specification requirements and would not be adversely affected by increased air content. This is likely the case for most all deck projects cast in California.

Recommendation:

- Specify air entrainment of 6.0 to 8.0 percent for all bridge deck concrete regardless of exposure conditions.

Casting Temperatures (Construction)

For most decks, placing cooler concrete during cooler weather can reduce the risk or severity of cracking.

- Springenschmid and Breitenbucher [1998] found that decreasing the casting temperature from 77 to 54°F (25 to 12°C) (decrease of 23°F/13°C) decreased the cracking temperature (the temperature of the concrete, after it has reached its peak temperature and is cooling, when cracking occurred) by 21 to 34°F (11 to 16°C), reducing the risk of cracking. As a general rule, they found increasing the concrete temperature at placement by 18°F (9°C) increased the cracking temperature 23 to 27°F (11 to 15°C), increasing the risk of cracking. The cracking temperature typically decreased more the amount the casting temperature was reduced, suggesting that lower concrete casting temperatures will reduce the effects of residual thermal stresses. Therefore, it is important to limit the concrete temperature at placement as it can have a significant effect on cracking.

- Our analysis found that decreasing the initial concrete temperature 10°F (5°C) approximately decreased the peak temperatures and maximum temperature differentials about the same amount (10°F/5°C). This decrease is especially beneficial with decks of box girder bridges because of their very slow cooling, which results in high residual temperatures being locked into the concrete for many days, when the concrete is very stiff already and has lost much of its creep potential.

- Kansas limits the temperature of plastic concrete to 50 to 75°F (10 to 24°C) at the time of placement.

- Pennsylvania reduced maximum allowable concrete temperature at placement from 75 to 80°F (24 to 27°C) to avoid deck cracking.

- Casting concrete during very cold or hot weather can worsen cracking. [Krauss and Rogalla, 1996] Ideally, concrete should be placed when air temperatures are between 40°F and 80°F (4 to 27°C). For most decks, placing cooler concrete during cooler weather can reduce the risk or severity of cracking.

To prevent large early thermal stresses and plastic shrinkage cracking, delivered concrete temperatures should be at least 10°F to 20°F (5 to 10°C) cooler than ambient air temperature when possible, especially when air temperature is 60°F (16°C) or greater. Concrete suppliers can wet or shade aggregates before mixing and chill mix water or replace part of the mix water with ice to reduce concrete temperatures. Ice must have sufficient mixing time to fully melt and disperse. Concrete suppliers routinely cool aggregates and fresh concrete batches with nominal additional cost.

Review Panel members were not in agreement with limiting the concrete temperature to 10 to 20°F (5 to 10°C) cooler than the ambient air temperature during warm weather based mostly on practical concerns on how to enforce such a specification. They were also not in agreement in limiting concrete temperatures to between 55 and 70°F (13 and 21°C). Our analysis clearly indicates that casting concrete at lower temperature will reduce the cracking risk in Caltrans structures. Therefore a means should be established to limit concrete temperatures for deck placements.

Recommendation:

- Limit plastic concrete temperature to no greater than 75°F (24°C) at the time of placement.

Secondary Recommendations

The recommendations provided in this section are considered Secondary Recommendations that are beneficial but may be less effective or important than the Primary Recommendations.

Aggregate Quality, Type, and Gradation (Concrete)

Aggregate quality and gradations limits are specified by Caltrans. Coarse aggregate limits for bridge decks include 1½-inch by ¾-(38 mm by 19 mm) by inch maximum size but smaller sizes are allowed.

Increasing the aggregate content beneficially reduces the paste content. This is generally beneficial for both cost and reducing cracking. Concretes with higher aggregate contents and lower cement paste contents are less likely to develop cracks. Leaner mixes are also thermally less expansive and develop smaller thermal stresses. The concrete mix should contain the largest practical aggregate size. Larger aggregates permit a leaner mix, help maintain workability, and reduce thermal and shrinkage stresses.

Well-graded, large aggregate can also reduce concrete shrinkage and bleeding or settlement that can initiate cracks over the reinforcing. The maximum aggregate size could be up to either one-third the deck thickness or three-fourths the minimum clear spacing between reinforcing bars, whichever is smaller, per ACI 318 and AASHTO requirements.

Local aggregates are typically used for deck concrete. Most local aggregates in California are siliceous and may be river run gravels, crushed stone, or blends. Siliceous aggregates tend to have higher coefficients of thermal expansion, and as such, concretes made with these aggregates tend to have larger thermal strains and stresses for a given temperature change and concrete stiffness. Because aggregates are the largest portion of a concrete mix, the thermal properties of the aggregate primarily control the thermal properties of the concrete. Thermal deformations can be reduced by using aggregates with lower expansion properties (CTE). While limestone aggregates have lower CTE values and lower thermal conductivity than siliceous aggregates making them preferred to reduce the risk of deck cracking, they are not commonly available in California. Crushed aggregate can produce a concrete with higher tensile strengths to resist cracking as long as water is not added to account for loss of workability.

- Deshpande, Darwin and Browning [2007], have shown that an increase in aggregate content (and reduction in paste content) leads to decreased shrinkage.

- French et al., [1999] found that increasing the aggregate quantity and lowering the cement content reduced cracking in Minnesota bridges.

- Springenschmid and Breitenbucher [1998] found that larger coarse aggregates tend to reduce concrete tensile strengths that may offset the benefit of the reduced paste content and shrinkage somewhat.

- Optimized aggregate gradations improve workability, reducing the amount of paste required for workability. Aggregate gradation by itself does not directly influence deck cracking, but the lower paste content that typically corresponds with an optimized aggregate gradation beneficially does. Furthermore, a mix with an optimum aggregate gradation has a reduced risk of segregation and is more cohesive, facilitating consolidation and reducing the risk of settlement cracking. Various gradation optimization techniques have been developed including the modified coarseness factor chart (Shilstone), percent retained plot, and the modified 0.45 Power chart. Kansas University developed the software program KU Mix to optimize aggregate gradations to reduce concrete shrinkage and deck cracking; it is available from their web site for no cost. An optimized "haystack" shaped percent retained gradation curve should reduce paste content and the risk of cracking. The Kansas Special Provisions [2007] for Low-Cracking High Performance Concrete Aggregates provides the following gradation requirements:

Grading Requirements for Mixed Aggregates for Concrete Bridge Decks (LC-HPC)

Type	Usage	Percent Retained on Individual Sieves - Square Mesh Sieves										
		1½"	1"	3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100
MA-4	Optimized for LC-HPC Bridge Decks*	0	2-6	5-18	8-18	8-18	8-18	8-18	8-18	8-15	5-15	0-5

*Use a proven optimization method, such as the Shilstone Method or the KU Mix Method.
Note: Manufactured sands used to obtain optimum gradations have caused difficulties in pumping, placing or finishing. Natural coarse sands and pea gravels used to obtain optimum gradations have worked well in concretes that were pumped.

- Aggregate type can have a large effect on the concrete shrinkage, with low-absorptive aggregates having a high elastic modulus (stiffness) that restrains shrinkage in concrete [Deshpande, Darwin and Browning, 2007]. The thermal properties of the aggregates directly affect stresses in the concrete due to temperature changes. Limestone aggregates typically have lower coefficient of thermal expansion (CTE) values and lower thermal conductivity than siliceous aggregates, making them preferred to reduce the risk of deck cracking.

- Alexander [1996] found that concrete shrinkage was affected primarily by the amount of water needed to attain a workable mix and the stiffness of the aggregate. Aggregate with a high elastic modulus tended to produce less shrinkage in most but not all cases.

- Saturated lightweight aggregate can be used to provide moisture to assist with cement hydration (internal curing) [Reynolds et al., 2009]. This technique requires specific planning and coordination to be effective.

It is likely not practical to restrict the use of aggregate types or sources for bridge decks in California to reduce the risk of deck cracking. However, the aggregate type can have a large effect on the concrete shrinkage and thermal properties, with low-absorptive aggregates typically having a high elastic modulus (stiffness) that reduces (restrains) shrinkage in concrete.

In general, concrete made with manufactured sands require more water and are more difficult to pump and consolidate. Concrete with aggregate types having high shrinkage or poor workability should be controlled by specifying a restrictive limit on concrete shrinkage and by requiring the contractor to complete a qualification test slab prior to deck placement to demonstrate pumping ability (if proposed), consolidation, and finishing. Using crushed coarse aggregate or crushed coarse aggregate blends can increase tensile strength and reduce cracking tendency. The effect of this is likely minor and it is not considered practical to require crushed aggregates statewide. Avoid crushed fine aggregate when it may affect workability adversely.

The Review Panel generally agrees that adjustments to aggregate size distribution will reduce the paste volume and improve workability. This also moves closer to a performance specification instead of the current prescriptive specification. All Review Panel members agree that increasing the maximum aggregate size is a good idea and easily done.

Saturated lightweight aggregate can be used to provide moisture to assist with cement hydration (internal curing). This technique requires specific planning and is considered beyond the scope of these recommendations.

Recommendations:

- Specify at least 1½ or 2 inch maximum aggregate size where available and feasible for the cover and reinforcement spacing up to either one-third the deck thickness or three-fourths the minimum clear spacing between reinforcing bars, whichever is smaller. In all cases use at least a 1 inch maximum aggregate size (No.57) gradation. Note, that 2-in. maximum-sized aggregate may not be available in many parts of California.

- Require that the Contractor provide results of an aggregate gradation optimization technique such as the modified coarseness factor chart (Shilstone), percent retained plot, the modified 0.45 Power chart or preferably the Kansas University software program KU Mix to optimize aggregate gradations. Paste content and shrinkage data should accompany this submittal.

- Require the Contractor to demonstrate that he can pump (if proposed), place, and cure the proposed mix adequately in a trial slab placement without cracking.

Research Idea: Perform research on the use of saturated lightweight aggregate to promote internal curing to reduce cracking risk.

Free Shrinkage and Shrinkage Reducing Admixtures (Concrete)

Caltrans limits the free shrinkage of deck concrete to 0.045% (450 microstrain) at 28 days. The concrete from the two bridges evaluated as part of this study had 28-day shrinkage values of 0.065 and 0.048 percent for field cured samples and 0.038 and 0.037 percent for laboratory cured samples. Shrinkage reducing admixtures (SRAs) are not routinely used by Caltrans for deck concrete.

- Babaei and Purvis [1996] concluded that to limit the average crack spacing to 30 ft. (9m) or more, the 28- day free drying shrinkage measured in the laboratory had to be less than 0.040 percent and the ultimate free shrinkage should be less than 0.070 percent. Lowering the ultimate applied free shrinkage from 0.070 percent to 0.050 percent will reduce shrinkage stresses by 29 percent if the effective elastic modulus and creep do not change.

- Lindquist, Darwin and Browning [2008] performed free-shrinkage tests to evaluate various concrete mixtures, and evaluated the construction and performance of various bridge decks in Kansas built with LC-HPC. Free shrinkage in concrete wet-cured for fourteen days was only slightly less than in those wet-cured for seven days. The study found that the addition of a shrinkage-reducing admixture (SRA) significantly reduced the free shrinkage, cement type did not have a great effect on free shrinkage, and the addition of Class F fly ash increased early-age shrinkage for concrete with either low- or high-absorption aggregate. The early-age effects of silica fume and slag were mixed and depended upon the aggregate used; shrinkage was generally reduced when these concretes were cured fourteen days verses only seven days.

- Many researchers have found that the use of SRAs in concrete reduced the shrinkage and cracking tendency [Shah et al., 1992; Brown et al., 2001; Tritsch et al., 2005; Brown et al., 2007]. Weiss et al., [2002; 2003] stated that SRAs significantly enhanced the cracking resistance of concrete by reducing the rate of shrinkage and the overall magnitude of shrinkage. SRAs reduced the surface energy of the water so there is less tension to make the concrete shrink. However, research [Folliard and Berke, 1997; Weiss et al., 2003] also found that SRAs may cause a slight decrease in the compressive strength of concrete although this research is mixed. A recent study financed by the Washington DOT [Zhuang, 2009] concluded that SRAs significantly reduce free shrinkage in concrete mixes that contain aggregate available in Washington State. SRA's can reduce the early-age and overall shrinkage about 50 percent. [Nmai et al., 1998, Gettu et al., 2007]

- A study conducted by Mora et al., [2009], shows that the reduction of the surface tension of the mixing water is an effective way for decreasing plastic shrinkage cracking. In this study, conventional and high strength concretes with superplasticizers and SRAs were exposed to drying in the plastic state. Continuous monitoring of the surface displacement facilitated the identification of the different stages of plastic shrinkage cracking. Measurements of capillary pressure, settlement, internal temperature and evaporation rate were also made. The results show the effectiveness of SRAs in reducing plastic shrinkage cracking, even in high strength concrete. This is attributed to the reduction in the evaporation rate, delay of the peak capillary pressure due to the development of menisci in the pores and lower settlement.

- SRA's can be applied topically to concrete surfaces by brushing or spray application before final concrete set at typical rates of 2.6 to 3.9 fl oz/sq. yd. [Nmai et al., 1998] Since the SRA applied in this manner is concentrated in the near surface of the concrete, it can be effective at reducing drying shrinkage gradients. Topical application of SRAs is not commonly done and would require further development and research.

Reducing shrinkage by decreasing paste contents or changing aggregate sources may result in an increase in the elastic modulus and decrease in concrete creep. Therefore, the reduction in shrinkage stresses in the deck will typically be less than the reduction in free shrinkage. Additionally, the higher elastic modulus and less creep may adversely affect early thermal stresses, which are typically larger in box girder bridges since the deck soffit typically cools much slower than a deck formed between steel or concrete girders.

Review Panel members agree that lowering shrinkage is a good idea but were concerned that doing so may be difficult in some areas and may increase costs. Lowering the 28-day maximum shrinkage to 0.040 percent was suggested instead of 0.035 percent. While this is still a high value it should be easily achievable especially with the use of aggregate gradation optimization techniques.

Review Panel members generally agree that the use of shrinkage-reducing admixtures should be allowed. However, maintaining the required air-void content may be a challenge when using SRA's, and they should only be used if an adequate air-void system can be achieved. This would be especially important in California for bridges where freezing occurs.

The primary disadvantage of using shrinkage reducing admixtures is increased cost and the need for increased quality control testing of the concrete. Inadequately mixed or improperly dosed SRAs can result in poor performance. Most agencies have not used SRAs routinely for all deck structures but, when used, will use them on priority deck projects.

Recommendations:

- Lower the maximum free shrinkage limit for deck concrete to 0.040 percent at 28 days.
- Evaluate the use and cost effectiveness of SRA's in deck concrete for priority projects. Require increased quality control of air void properties on these projects.

Research Idea: Evaluate the effectiveness of SRA's in reducing deck cracking in several demonstration projects. Investigate surface-applied SRAs.

Chemical Admixtures, Water/Cement Ratio, and Water Content (Concrete)

Caltrans limits water content based on the total cementitious content submitted. Concrete penetration (slump) is also specified; however, Section 90 allows if Type F or Type G chemical admixtures are added to the mix, the penetration requirements shall not apply and the slump shall not exceed 9 inches after the chemical admixtures are added.

- Krauss & Rogalla [Transverse Cracking in Newly Constructed Bridge Decks, 1996] found that total water content or water-cement (w/c) ratio did not have a direct relationship on cracking. This was more recently confirmed by Deshpande, Darwin and Browning [2007] and French et al., [1999]. Paste content rather than water content controls cracking. Further, w/c ratio plays a minor role in shrinkage compared to paste content.
- However, concrete water cement (w/c) ratio directly affects concrete strength, modulus and creep, with higher w/c ratios being favorable to reducing deck cracking. Kansas requires a w/c of between 0.43 to 0.45, and Pennsylvania increased the w/c of deck concrete from 0.40 to 0.43 to reduce deck cracking. Research has also shown that autogenous shrinkage increases with w/c ratios below 0.40. Strength and permeability can be a problem over 0.45.

High-range water reducing admixtures (HRWRAs) often accelerate stiffness gain and reduce creep potential, thereby increasing the risk of deck cracking. When used to increase slump, HRWRAs can increase the risk of settlement cracking over supported deck reinforcing. Set accelerators can worsen bridge deck cracking by accelerating and increasing temperatures during hydration, increasing early stiffness and reducing early creep, and increasing early shrinkage.

Careful use of retarding admixtures is needed. Set retarders are often used to allow continuous deck placement. Many retarders merely shift when hydration substantially starts and do not substantially affect the rate of hydration after the initial start. Retarders can increase the susceptibility of the concrete to plastic shrinkage and settlement cracking and should be avoided without adequate testing.

Chemical admixtures for concrete can individually increase concrete shrinkage by 35 percent and still meet ASTM C494 specifications. Combinations of approved admixtures can increase concrete shrinkage even more. Admixture suppliers should be required to submit independent shrinkage test data for chemical admixtures used for deck concrete; admixtures that significantly increase shrinkage should not be used. One practice is to require all allowable water to be added to concrete mixes before the use of water-reducers is allowed.

Review Panel members generally agreed that avoiding chemical admixtures that increase shrinkage is a good idea, but they favored a performance shrinkage limit instead of limiting admixtures. This requires testing of each mixture at the highest dosages of admixtures to be used and retesting anytime admixtures are changed.

Review Panel members generally agree with limiting w/c between 0.42 and 0.45 but were concerned that 0.42 may be too low. Review Panel members generally disagreed with a recommendation that all mix water be added at the plant and that only admixtures be allowed to be used at the site to adjust workability.

Recommendations:

- Mixes should be tested for shrinkage at the maximum admixture dosages that may be used. Mix design approvals must be based on actual admixtures to be used and substitutions should not be allowed without shrinkage test data. Do not allow chemical admixtures that increase free shrinkage. Use only water reducing admixtures that do not increase shrinkage and limit their dosages if used.

- Specify and maintain the in-place concrete water/cement ratio between 0.43 to 0.45.

- Do not allow concrete accelerators to be used in deck concrete. Test concrete retarders to ensure that they do not increase heat of hydration temperatures. Use immediate fogging and continuous moist curing if retarders are used.

Research Idea: Require chemical admixture suppliers to submit independent test data for free shrinkage. Modify approved admixture list as appropriate.

Concrete Slump and Settlement (Concrete and Construction)

Caltrans specifications Section 90 allows if Type F or Type G chemical admixtures are added to the mix, the penetration requirements shall not apply and the slump shall not exceed 9 inches after the chemical admixtures are added.

Reinforcing size, concrete cover, and concrete slump affect cracking from settlement. Higher slumps tend to aggravate settlement cracking. Settlement cracks and voids decrease the effective cross-section that can resist tensile stresses from thermal, shrinkage and load effects, thereby increasing the risk of full-depth cracking even at low stress levels.

- Kansas requires a design slump range from 1½ to 3 inches, with a maximum allowable slump of 3½ inches (89 mm) during placement.

- Pennsylvania reduced deck cracking when they decreased the target slump from 6 to 4½ inches (152 to 114 mm).

Most concrete bridge decks are well consolidated. Inadequate consolidation can increase settlement-related cracking over reinforcing, so all deck concrete must be thoroughly consolidated with mechanical vibration. Thorough internal vibration is needed. Ganged sets of internal vibrators mounted to the deck finishing machine can improve the thoroughness of internal vibration.

After concrete is placed, the aggregates in the fresh concrete settle and bleed water comes to the surface. Obstruction to this settlement by the supported reinforcement can cause voids to develop immediately below the bars and cracking to occur over the bars. In severe cases, these cracks have the characteristics of plastic shrinkage cracks but follow over the reinforcing bars. Internal cracks that are not visible on the surface could also occur due to settlement, and such cracks could later propagate to the surface and through the deck thickness.

Large diameter bars, close to the surface (low concrete cover), and high slump, and slow setting concrete aggravate settlement cracking. Using a well graded, low slump concrete, with proper consolidation can prevent settlement cracking.

Cracks most common to bridge decks are typically transverse and directly in-line with the top mat reinforcing steel but are full depth. Plastic settlement in decks could result in fine hairline cracks that are not easily visible on the surface but be sufficient to initiate full depth cracks at later ages. Therefore, efforts to minimize settlement cracking are important. These include placing low-slump concrete, providing adequate concrete cover to the top reinforcing, adequately consolidating the concrete, and using the minimum diameter and amount of transverse reinforcement.

While some Review Panel members felt slump control is important, others were concerned that limiting slump to 1½ to 3½ inches (38 mm to 89 mm) is too strict and could affect placement issues such as consolidation and pumping. They also recommended changing the allowable slump based on the type of water-reducing admixture used.

Recommendations:

- Limit slump of concrete to a maximum penetration of 2 inches (51 mm) maximum slump of 4 inches (102 mm), except when mid- or high-range water reducers are used then specify a maximum penetration of 2 1/2 inches (64 mm [5 inch slump]).

- Provide adequate consolidation to the fresh concrete using thorough internal vibration.

- Use the minimum diameter and amount of transverse reinforcement.

Cement (Concrete)

Caltrans requires the use of Type II or Type V cement. For Type II cement, the C_3S content shall not exceed 65 percent. It also allows blended cements (AASHTO M 240) without limiting the pozzolans content. Cement alkali is limited to 0.60 percent.

- Burrows [1998] recommended to reduce deck cracking that cements should have a low C_3S content of less than 45 percent, below alkali (less than 0.6 percent Na(eq)), and be of coarse grind (less than 320 m^2/kg). Type II or V cements are preferred over Type III high early cement.

- Chariton and Weiss [2002], Deshpande et al., [2007], and Burrows [1998], found that concrete cast with coarser cements shrank less than concrete containing finer cements. Large cement particles likely provide internal restraint to shrinkage and the lower capillary stresses due to the coarser pore structure. They also typically will reduce the rate of hydration, thereby reducing peak temperatures and thermal stresses. However, some coarser cements may increase the permeability of the concrete slightly.

Cement fineness and chemical composition affect the rate of hydration, early strength gain, and the heat generated initially by the concrete. Modern cements are more apt to cause cracking because they are finer, set up faster, generate more heat, and have higher sulfate and alkali contents [Gebhardt]. Use Type II cement instead of Type I to reduce peak temperatures during early concrete hydration and develop smaller corresponding thermal stresses. Type IV (low heat-of-hydration) cement would be a good choice but it is no longer manufactured in the United States. Type III (high-early-strength) cement should not be used because it increases early temperatures and related stresses, and it reduces beneficial creep. Differences within Type II, low alkali cements currently specified by Caltrans likely do not have a significant influence on deck cracking, and specifying coarse ground cements or specialty portland cements that are less prone to cracking is not practical statewide at this time.

Recommendations:

- Continue to use current practice.
- Do not allow substitutions of high early strength (Type III) cements or high alkali cements.
- If choices in cement are available, use cement with a low C3S content (less than 45 percent) and coarsely ground (less than 320 sq. m/kg).

Casting Time (Construction)

Caltrans does not limit when deck placement can occur.

- Our analysis of Caltrans structure types showed the largest temperature gains and the largest temperature differentials, which adversely affect the risk of cracking, occurred when the concrete was placed in the very early morning. Conversely, the smallest temperature gains and differences occurred when the concrete was placed between mid-afternoon and early evening. Casting the temperature early in the morning results in the environment being warmer than the concrete and solar radiation being greatest, just when the hydration begins to develop, maximizing the rate of heat gain. Conversely, when the concrete is cast in the afternoon and early evening, the air is cooler and there is little or no solar radiation several hours when significant hydration is developing, reducing temperatures; the air will be cooler than the concrete the next afternoon, so that the air is almost always cooler than the concrete. The effects of casting time are greatest on decks of box girder bridges, because of the reduced cooling they typically have.

Recommendation:

- Whenever possible and especially during hot weather, require deck placement to occur in late afternoon and evening, after 3 p.m.

Concrete Testing and Quality Control (Construction)

To avoid deck cracking, the Contractor and project staff need to understand the significant issues related to the causes of deck cracks and clear communication is needed between all parties.

- Kansas requires mandatory pre-bid conferences with contractors to discuss requirements for low cracking (LC-HPC) bridge decks. They also require contractors to submit a Quality Control plan detailing procedures for controlling evaporation rate. Preconstruction meetings are held with contractor to discuss cracking prevention. After the qualification batch of concrete has been approved, the construction of a qualification slab is required 15 to 45 days prior to placing concrete in the bridge deck. The slab must be identical in geometry as the deck, but does not need to be elevated. The methods, equipment, crews and concrete are required to be the same as for the placement of the bridge deck. Finally after construction, a post-construction conference is held to discuss successes and problems on the project.

- Pennsylvania suggests review and improvement of quality control and quality assurance operations to control concrete and construction. South Carolina performs trial mixes of bridge deck concretes to evaluate for cracking.

- Much research has been done on evaluating the cracking tendency of various concrete mixtures in restrained concrete tests [ACI 231R-10, 2010]. Such testing of the concrete mix performance under restraint is relatively inexpensive and most jobs have sufficient lead time of several weeks to allow for such testing of proposed concrete mix designs.

Pre-job meetings to discuss measures to prevent deck cracking, qualification test slabs, and clear communication between all parties are needed. Specifications need to be reasonable and enforced. QC testing requirements and expected actions in the event of failed tests should be reviewed with QC staff so they know what to do when concrete tests do not meet specifications. Failed tests should initiate an increased testing frequency.

A continuous delivery of concrete is important and conflicts occur when QC testing slow production. Making the Contractor primarily responsible for concrete QC testing, employing independent, certified test laboratories, reduces any delays to concrete delivery and installation. Caltrans testing staff should be responsible for oversight and QA split sample testing of a percentage of the independent lab tests. This allows the Contractor to control concrete supply while performing adequate quality control testing.

Review Panel members did not fully agree on the value of the construction of a qualification slab, especially if the contractor is experienced.

Recommendations:

- Hold pre- and post- pour meetings with the Contractor to discuss prevention of deck cracking and to obtain feedback.

- Have the Contractor submit a cracking mitigation and curing plan.

- Make the Contractor responsible for quality control of plastic concrete and perform quality assurance (QA) split sampling to avoid slowing concrete placement.

- Post-construction meetings should be held with all parties involved to gather and document opinions on the success or problems related to the bridge deck placement.

Research Idea:

- Require deck mixes to be tested for retrained cracking tendency in addition to free shrinkage. Develop a data base of the cracking tendency of the various mixtures.

Bridge Geometry and Deck Reinforcing (Design and Construction)

Stresses develop in a bridge deck primarily because of the history of temperature and shrinkage differences within the deck itself, and differences between the deck and the structural elements below it. As the new concrete deck hardens and stiffens, it is typically much warmer than the lower portions of the bridge section that will restrain the new deck concrete. As the new deck concrete cools, the contraction is similar to that from drying shrinkage, and is additive to the shrinkage strains that develop. This contraction or shortening of the deck tends to make the girder curve to a concave-upwards position. Interior supports then push upwards against the bridge to keep the bridge position at the supports in its original vertical position. This creates negative bending in the bridge over the supports, resulting in tensile stresses in the deck that can cause or contribute to cracking. Traffic has a similar effect in the bridge over the supports, with additional adverse tensile stresses developing in the bridge deck over the interior supports. For a single-span bridge, in contrast, traffic beneficially compresses the deck.

Generally, decks placed over stiffer girders or stiffer lower box sections will have more restraint, will develop larger thermal and shrinkage stresses, and be more prone to transverse cracking. These effects typically vary less for concrete box girders.

- French et al., [1999] studied seventy-two bridges in the Minneapolis/St. Paul metropolitan area. They found simply-supported prestressed concrete girder bridges in good condition compared to continuous-span steel girder bridges. The degree of restraint was one of the dominant design factors related to deck cracking. Another factor thought to affect transverse deck cracking was the size and spacing of the transverse top reinforcing bars. A trend for improved condition rating was found with increased bar spacing, and smaller bars (No. 5) had less cracking than larger bars (No. 6). This effect was not seen on prestressed girder bridges and the size and spacing of the longitudinal reinforcing did not have a significant effect on transverse cracking in steel or concrete girder bridges.

- Frosh [2009] reported that reinforcing did have an effect on cracking. Larger transverse bars more closely spaced were found to increase transverse cracking; one explanation for this could be the increased incidence of microcracks and voided areas at reinforcing due to settlement of plastic concrete.

- AASHTO LRFD covers beneficial use of smaller bar sizes to control cracking.

- ACI Committee 224 indicate that the normal reinforcement percentage of between 0.18 to 0.20 percent is not adequate to control crack widths to prevent water ingress, and that the percentage should exceed about 0.60 percent to more appropriately control crack widths.

When designing continuous span bridges, for the regions above the interior supports, the designer should evaluate the negative moments caused by early temperature differences and subsequent cooling, and by differential shrinkage within the girder. In lieu of performing detailed finite element analyses of these temperatures and strains, the designer can conservatively assume that the concrete deck is at a stress-free condition when it reaches its maximum temperatures, with a uniform temperature gain approximately equal to the adiabatic heat gain of the concrete, and that all cooling occurs with the concrete at its full elastic modulus. A similarly conservative assumption for assessing shrinkage effects would be to assume that all shrinkage occurs after the concrete has reached its full stiffness. Additional longitudinal reinforcement should be added to the deck as necessary to resist the negative moments above the supports caused by early temperatures and long-term shrinkage, placing the reinforcement closely together to control cracking. For a given reinforcement cross-section area, smaller bars spaced more closely together control cracking than larger bars space farther apart (longitudinal bars may be most important).

Increasing the longitudinal reinforcement will typically increase the number of transverse cracks but decrease the crack widths. However, simply adding steel, without adding enough to keep cracks so tight as to prevent water intrusion into the cracks, could potentially make deck durability worse since more cracks may occur due to the added restraint. Adding deck reinforcement to keep cracks tight enough to prevent ingress of water and deicers is likely not practical or cost effective. Further, steel congestion issues could limit the maximum concrete aggregates size used to reduce paste and shrinkage, an important factor to reduce cracking.

Review Panel members generally agreed that reducing the size and amount of transverse reinforcing steel is a good idea.

Recommendations:

- Prefer simply-supported bridges over continuous-span bridges.
- When continuous span bridges are designed, consider the negative moments caused in the girders above the supports, and provide additional reinforcement to strengthen the section and control cracking.
- Use the minimum diameter and minimum amount of transverse reinforcement. Institute on a job-to-job basis.
- Provide more closely spaced longitudinal reinforcement where applicable.

CLOSURE

Early-age cracking of bridge decks is a nationwide problem that is a result of many complex interactions and eliminating deck cracking has proven to be very difficult. The researchers at Wiss, Janney, Elstner Associates, Inc. (WJE) appreciate this opportunity to work with the California Department of Transportation (Caltrans) to meet the objectives of this project and to recommend changes in current Caltrans design and construction specifications, and construction procedures to mitigate early-age cracking in bridge decks in California.

We would like to thank all the Caltrans and industry people that participated in this project with special thanks to Madhwesh Raghavendrachar and Anthony Gugino of Caltrans for their guidance and input. We also greatly appreciate the help of Boris Stein from Twining Laboratories and Dr. John Bolander, professor at the University of California, Davis, for their assistance with this project.

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Appendix A Concrete Laboratory Testing

Appendix B Field Monitoring Data

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Appendix C Lattice Modeling